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I. INTRODUCTION

1.1 SCOPE

Previous issues of "Bridge Design Practice" have not treated foundation design as a separate topic but contain bits of information under various headings.

This section is an attempt to concentrate the material on substructures and foundations and to apply the Load Factor Design method to their design.

The method consists of applying factored loads (AASHTO Article 1.2, Table 1.2.22A) to ultimate capacities of foundation elements which have been modified by a strength reduction factor (ϕ).

In applying group loadings to bent foundations, a method of applying seismic loading is presented which is consistent with the current philosophy of seismic analysis and design.

1.2 FOUNDATIONS (GENERAL)

The substructure is that part of the structure which serves to transmit the forces of the superstructure and the forces on the substructure itself onto the foundation.

The foundation is that part of a structure which serves to transmit the forces of the structure onto the natural ground.

If a stratum of soil suitable for sustaining a structure is located at a relatively shallow depth, the structure may be supported directly on it by a spread foundation. If the upper strata are too weak, the loads are transferred to more suitable material at greater depth by means of piles or piers.

The design of the structural elements for foundations, substructures and retaining walls is in accordance with the provisions of AASHTO.

The design of the structural elements is well codified; the soil mechanics aspect of the design is not codified to any extent.

The bearing capacities of foundation soils, settlements, the ability of piles to transfer load to the ground, lateral earth pressures and lateral earth resistances are some of the items which are determined by evaluation of site investigations and/or current practice.

In stability analyses the factors of safety for overturning and sliding are not specified in AASHTO. Determination of values to be used is based on accepted practice and evaluation of the risk involved. Part III will enumerate values currently used.

II. STRUCTURE FOUNDATIONS

2.1 CAPACITY OF SHALLOW FOUNDATIONS

A shallow foundation is a term applied to a footing having a depth to base width ratio of less than or equal to 1. ($D_f/B \leq 1$)
Where depth D_f = the distance from the ground surface to the contact surface between the soil and the base of the footing.
B = width of footing.

Two things control the capacity of a shallow foundation:

1) the ability of the soil to support the loads imposed upon it, known as the bearing capacity of the soil.

2) the amount of total or differential settlement that can be tolerated by the structure being considered.

2.1.1 Ultimate Bearing Capacity of Soil

When a load is applied to a limited portion of the surface of a soil the surface settles. The relation between the settlement and the average load per unit area (q_d) is represented by a settlement curve (Fig. 2-1). If the soil is dense or stiff the curve is similar to C_1. The abscissa q_d of the vertical tangent to the curve represents the ultimate bearing capacity of the soil. If the soil is loose or fairly soft, the settlement curve may be similar to C_2 and the bearing capacity is not always well-defined. The bearing capacity of such soils is sometimes assumed to be equal to the abscissa q'_{dA} of the point at which the settlement curve becomes steep and straight. A more conservative value is to use the bearing capacity at the abscissa q'_{dB} at the point where the settlement curve C_2 ceases to be linear.

When the bearing capacity of a real footing is exceeded the soil fails along a surface of rupture similar to $fede_1f_1$ in Fig. 2-2(a). An approximate method of evaluating the ultimate bearing capacity consists of equating separately the following three components: See Fig. 2-2.

1. The cohesion and friction of a weightless material carrying no surcharge.

2. The friction of a weightless material upon addition of a surcharge q on the ground surface.

3. The friction of a material possessing weight and carrying no surcharge.

The approximate equation for bearing capacity of a shallow foundation is:

$$q_d = cN_c + \gamma D_f N_q + 1/2\gamma BN_\gamma$$

q_d = bearing capacity per unit area

c = cohesion

γ = unit weight of soil

N_c and N_q are bearing capacity factors with respect to cohesion and surcharge respectively.

N_γ accounts for the influence of the weight of the soil.

All the bearing capacity factors are dimensionless quantities depending only on Ø.

Meyerhof's values for the bearing capacity factors are given in Fig. 2-3. Fig. 2-4 is a direct correlation between the bearing capacity factors and the N-values obtained from Standard Penetration Tests.

The solid lines in Meyerhof's table are to be used with firm soils corresponding to load settlement curve C_1 in Fig. 2-1.

The dash lines respectively are for soils which would correspond to curve C_2 in Fig. 2-1. These soils would not fail in quite the same manner as the firmer soils, and the footings would settle before shear became mobilized along the entire surface of Fig. 2-2.

For this local shear failure an approximate solution is to use 2/3 the value for both cohesion and friction, i.e.,

$$c' = 2/3\ c$$

$$\tan Ø' = 2/3 \tan Ø$$

the equation for bearing capacity becomes:

$$q_d' = 2/3\ c\ N_c' + \gamma D_f N_q' + 1/2\gamma\ BN_\gamma'$$

and the bearing capacity factors N_c', N_q' and N_γ' are taken from the dash lines using the angle of shearing resistance Ø'. (Fig. 2-3)

The basic equation for bearing capacity relates to a continuous or strip footing. Modifications of the formula are available for use with square, circular or footings of other shapes. (Ref. 1)

Tables are also available in the references for modifying the basic bearing capacity equation for the condition of a footing on or at the top of a slope. (Ref. 2)

2.1.2 Determination of Allowable Settlement

There are many methods of estimating the settlement of a shallow foundation. Some of these are:

1. Table, (Ref. 8) titled "Allowable Bearing on Granular Sediments".

2. Fig. 2-5

3. A Meyerhof relation:

Equation 1 $qs = N/8$ when $B < 4'$

Equation 2 $qs = N/12 \left(\frac{B+1}{B}\right)^2$ when $B \geq 4'$

Where qs = allowable bearing in tons per square foot

N = blow count obtained from Standard Penetration Test

ρ = settlement in inches

B = width of the foundation under consideration

Many other methods exist, however, these three all relate to the value N obtained from the Standard Penetration Test which is used by TransLab Engineering Geology for determining soil parameters in most cases.

If a footing is underlain by a layer or layers of compressible material, settlement due to the compressibility of the layers must be added to the amount of settlement obtained from the procedures noted above.

2.1.3 Design Procedures (General)

Historically the design of shallow spread foundations consisted of proportioning the footing to distribute service loads on the foundation soil such that the maximum bearing pressure did not exceed an allowable capacity as predetermined by TransLab Engineering Geology.

This allowable bearing capacity was that unit load which it was estimated would produce a maximum differential settlement of 1/2".

The allowable bearing capacity in no case, however, was to exceed the ultimate bearing capacity reduced by a factor of safety of 3.

Load factor design of shallow footing foundations will employ the use of the ultimate bearing capacity of the foundation soil.

TransLab Engineering Geology will now be providing values for ultimate bearing capacity of spread foundations and information as to the method by which they were determined.

2.2 CAPACITY OF DEEP FOUNDATIONS

A pier is a structural member of steel, concrete or masonry that transfers a load through a poor stratum onto a better one. A pile is essentially a slender pier that transfers a load either through its tip onto a firm stratum (point bearing pile) or through side friction onto the surrounding soil over some portion of its length (friction pile).

Load settlement curves for piers and piles are similar to those for footings. The definition of bearing capacity of piers and piles is identical with that of footings.

2.2.1 Bearing Capacity of Piers

Piers founded on firm soil beneath layers of more compressible material act more like spread footings with surcharge approximately equal to γD_f. The bearing capacity q_p may be determined using the proper form of the basic bearing capacity equation considering the shape of the pier.

If the soil surrounding the pier is homogeneous the shear patterns in the soil at failure are altered and the bearing capacity formulas no longer apply.

Consider a cylindrical pier of radius-r, and depth D_f. At failure the load is expressed as:

$$Q_d = Q_p + Q_s = q_p A_p + 2\pi r f_s D_f$$

q_p = the bearing capacity per unit of area of the soil beneath the base.

A_p = the base area.

f_s = the average value at failure of the combined effect of adhesion and friction along the contact surface between pier and soil. The latter term, commonly referred to as "skin friction".

The values for adhesion and friction can be determined approximately in the lab. However, the method of installation of a pier has a marked influence on the values.

The bearing capacity of a pier then is most reliably determined using empirical values for q_p and f_s as selected by someone experienced in evaluating conditions existing at the site and construction procedures.

Tables containing approximate values of the parameters (f_s,q_p) for various soils and conditions are available in Ref. 1.

2.2.2 Bearing Capacity of Piles

In general, the bearing capacity of a single pile is controlled by the structural strength of the pile and the supporting strength of the soil. The smaller of the two values is used for design.

Piles driven through soft material to point bearing may be dependent upon the structural strength of the pile for their bearing capacity.

The supporting strength of the soil is the sum of two factors - the bearing capacity of the area beneath the base, and the frictional resistance on the contact surface area for the length of the pile.

For point bearing piles the former is of primary significance while for friction piles the latter is of primary significance.

Structural sections of piles are to be designed using the provisions for the material being used and satisfying the minimum requirements specified in AASHTO and this section in foundations. Ref. 13 has standard designs for 45 ton and 70 ton piles which are laterally supported by soil. Memo to Designers 3-3 may be used for design of 15" diameter pile extensions.

Displacement of soil during installation of piles creates varying states of stress in the surrounding soil and makes computation of "skin friction" unreliable.

Ranges of empirical values for "skin friction" in various soils are in Ref. 1. Local experience is of great value in selecting empirical values to be used.

The Engineering News Record Formula is a Dynamic Formula for determining pile capacity. The formula has been in use in this country for some 90 years. The ENR formula, as with several similar dynamic formulas (Danish, Janbu, etc.), estimates bearing known as dynamic resistance from measuring the average penetration of the pile under the last few blows of the hammer. While the formula is theoretically sound the results obtained from its use are unreliable. Ref. 1 cites load tests on piles driven using ENR which had bearing capacities ranging from 1.2 to 30 times the value obtained by the formula. The formula itself has a built in factor of safety of 6. Ref. 1 and many other respectable soil texts recommend against the use of ENR for determining pile capacity.

The Wave Equation is a sophisticated Dynamic Pile Formula which is now being used on an experimental basis by the TransLab Engineering Geology.

A pile load test is probably the best method available for determining the bearing capacity of an individual pile. The tests are quite expensive, however, and on small jobs the cost of their use cannot be justified.

On large jobs where there are many piles to be driven, a pile load test is performed under the direction of the TransLab Engineering Geology. Details on the performance of a pile load test are available in Reference 8.

2.2.3 Design Procedures (General)

Pile foundations have historically been designed using service loads. TransLab Engineering Geology has until now recommended allowable bearing capacities of piles using a factor of safety of 2 against bearing failure.

The TransLab Engineering Geology will now furnish the ultimate bearing capacity of the pile and the method by which it was determined.

Pile capacity designations currently used will not be changed on contract documents, standard plans, etc. For example, the class 70 pile will be designed using the ultimate capacity of 140 tons (280 kips), but will still be designated a 70 ton pile.

2.3 DESIGN OF BENT FOUNDATIONS

Procedures for design of footings for columns. (Note: Procedures also applied to footings for pier walls in the longitudinal direction.)

Columns on Individual Footings

1. Determine column section requirements based on the Load Factor Design Group Loadings in AASHTO and using the design strength of the member. The design strength of a member or cross section equals the nominal strength modified by the strength reduction factors (Ø) specified in AASHTO. The nominal strength of a member or cross section equals the strength calculated using the specified compressive strength of the concrete and the specified yield strength of the reinforcement.

2. Determine as a minimum the nominal moment strength of the column in the direction of the principal axes of the footing at the locations where plastic hinges may form when the structure response to seismic loading causes inelastic action in the columns. These nominal moments strengths shall be those associated with the unfactored dead load axial force. Currently there is a TSO program titled "YIELD" which can be used to develop interaction diagrams for column sections. Also interaction diagrams for the standard column sections are available.

3. Determine the column probable plastic moments. The column probable plastic moments equal the nominal moment strengths increased by a factor equal to 1.30.

4. Using the column probable plastic moments, determine the corresponding column shear forces.

   Determine the axial forces in the columns due to overturning when the probable plastic column moments are developed. Using these column axial forces combined with the dead load axial forces, determine new column probable plastic moments. Using these new probable plastic moments determine the column shear forces. If the sum of these new column shears are not reasonably close (within 10 percent) to the sum of the previously determined column shears, reevaluate the column probable plastic moments and column shears.

5. The ultimate moments to be used for designing the footing shall be those that are the least critical of the following two cases:

   A. The final column probable plastic moments at the base of the column.

   B. The column moments at the base of the column from an elastic seismic analysis before any reduction for ductility (Z factor). Two orthogonal directions of earthquake motion shall be considered. The moments which result from the analysis of earthquake motion in one direction shall be combined with 30 percent of the moments which result from the analysis of earthquake motion in the other direction. This will result in applied moments acting in two orthogonal directions simultaneously. The two possible combinations of moments shall be considered. See Figure 1.

6. The horizontal force induced into the structure at the bent is limited to the column shear force associated with the development of the probable plastic moments. The lateral resistance of the footing may be considered adequate provided the material surrounding the footing and upper portion of the pile of pile footings has a standard penetration value, N, of at least 4. The piles for pile footings should be designed to sustain large induced curvatures and still maintain their design axial load.

7. The ultimate vertical forces to be used for designing the footing shall be the unfactored dead load force combined with the axial forces associated with the ultimate moments of Step 5.

8. Design a footing to resist the ultimate moments and forces of Steps 5 and 7. For resisting the vertical forces and moments use the ultimate soil bearing capacity or the ultimate pile

axial capacity using a strength reduction factor (Ø) equal to 1.0.

When determining the flexural capacity of the footing, use a strength reduction factor (Ø) equal to 1.0 and a yield strength of reinforcement equal to 1.0 times f_y.

When determing the shear capacity of the footing, use a strength reduction factor (Ø) equal to 0.85 and a yield strength of reinforcement equal to 1.0 times f_y.

9. Design the piles of pile footings to sustain large curvatures and the design axial force.

When determining the transverse reinforcement required in the piles, consideration shall be given to confining the core in those regions where plastic hinges may be expected to form. In these regions the minimum volumetric ratio shall be:

$$\rho_s = 0.12 \frac{f'_c}{f_y}\left(0.5 + 1.25 \frac{P_e}{f'_c A_g}\right)$$, except ρ_s need not be greater than 0.012.

(Generally, where the soil strata below the footing increase in strength with depth the plastic hinge in the piles can be assumed to form at the footing. For this case the confining reinforcement shall extend not less than twice the longest cross-sectional dimension of the pile or 36 inches, whichever is greater.)

The minimum recommended transverse reinforcement in the top 6 feet of pile 20" Ø or less shall be equivalent to a W6.5 spiral at 3 inch pitch, the minimum transverse reinforcement in the remainder of the pile shall be equivalent to a W6.5 spiral at 6 inch pitch.

When determining the axial tensile force resistance of the piles, use a strength reduction factor (Ø) equal to 1.0 and a yield strength of reinforcement equal to 1.0 times f_y.

When uplift capacity of the piles is required, verify with the TransLab Engineering Geology that the pile length specified is adequate for the design axial tensile force.

10. Check the footing design using the Load Factor Design Group Loadings in AASHTO, except omit Group VII. The ultimate soil bearing capacity shall be modified by a strength reduction factor (Ø) equal to 0.5 and the ultimate pile bearing capacity shall be modified by a strength reduction factor (Ø) equal to 0.75.

When checking the adequacy of the footing sections use the design strength of the member specified in AASHTO. Revise footing, if required.

11. Transverse column reinforcement shall be provided for confinement and shear resistance.

The cores of the column shall be confined by transverse reinforcement in the regions where plastic hinges are expected to form.

The extent of these regions shall be assumed to be length not less than (1) the maximum dimension of the column, (2) one-sixth of the clear height of the column, (3) 24 inches. For the flared end of a flared column the extent of the plastic hinge region shall be assumed to be a length equal to the flare length plus the greater length of (1), (2), or (3) above.

The transverse reinforcement for confinement within these regions shall provide the greater of the two following volumetric ratios for spirally reinforced columns:

volumetric ratios for spirally reinforced columns:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}\left(0.5 + 1.25\frac{P_e}{f'_c A_g}\right)$$

or

$$\rho_s = 0.12\,\frac{f'_c}{f_y}\left(0.5 + 1.25\frac{P_e}{f'_c A_g}\right)$$

The transverse reinforcement for confinement at any location within the column shall provide the following volumetric ratio for spirally reinforced columns:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}$$

It is recommended that all columns be spirally reinforced.

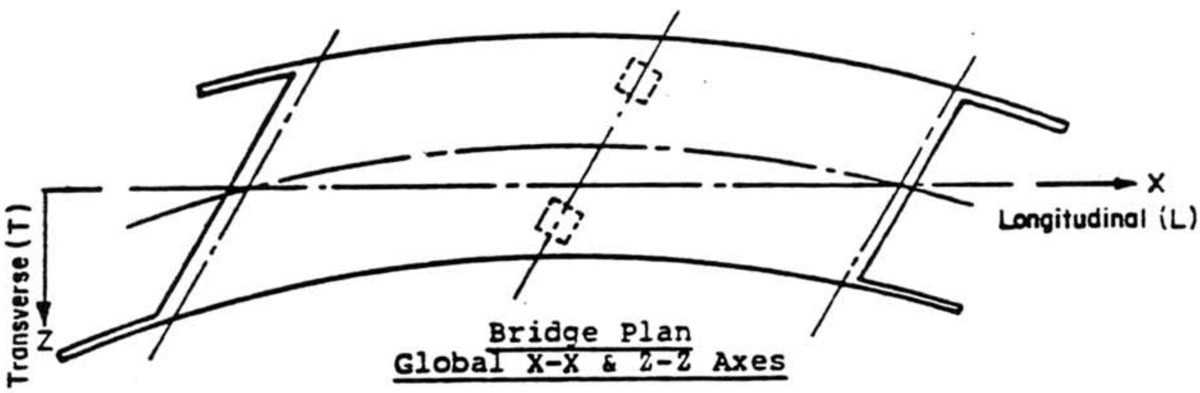
For those columns reinforced transversely with rectangular hoop reinforcement, refer to SEAOC's "Recommended Lateral Force Requirements and Commentary" for the required confinement reinforcement, and AASHTO.

The design column shear forces determined in Step 6 shall be resisted by concrete and transverse column reinforcement. In regions where plastic hinges may form, use the core section of the column to resist the shear force. In regions other than where plastic hinges may form, use the gross section of the column to resist the shear force.

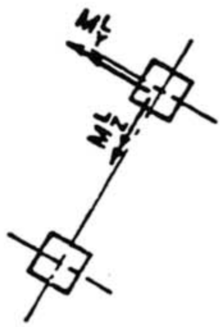
When determining the shear resistance of the column, use a strength reduction factor (ϕ) equal to 0.85 and a yield strength of reinforcement equal to 1.0 times f_y. If the transverse reinforcement required for confinement is also adequate for shear then no additional transverse reinforcement is required. The reinforcement requirements for confinement and shear are not additive.

12. For columns considered hinged at the top of the footing, the bottom of column design shear forces and design axial tensile forces shall be resisted by concrete area and vertical reinforcement according to the provisions of AASHTO.

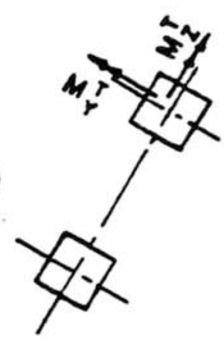
The column design axial compressive force shall be resisted by concrete area and vertical reinforcement according to the provisions of AASHTO. Vertical reinforcement provided to resist the column shear forces and the column axial tensile forces may be used in resisting the column axial compressive force.



Bridge Plan
Global X-X & Z-Z Axes



(Local Y-Y & Z-Z Axes)



Longitudinal Load Results	Transverse Load Results
Case 1	**Case 2**
$M_Y^D = M_Y^L + 0.3M_Y^T$	$M_Y^D = M_Y^T + 0.3M_Y^L$
$M_Z^D = M_Z^L + 0.3M_Z^T$	$M_Z^D = M_Z^T + 0.3M_Z^L$

where M_Y and M_Z are about local axes.

Combination of Orthogonal Seismic Forces

Figure 1

13. At each bent, individual footings may be connected by ties to distribute the total horizontal force in the plane of the bent to each footing in proportion to its capability to resist horizontal forces. The ties shall be capable of resisting in tension and compression the design ultimate axial force required to redistribute the total horizontal force.

    When determining the tensile capacity of the tie use a strength reduction factor (ϕ) equal to 1.0 and a yield strength of reinforcement equal to 1.0 f_y.

    When determining the compressive capacity of the tie use a strength reduction factor (ϕ) equal to 0.75, a concrete strength equal to f'_c and a yield strength of reinforcement equal to 1.0 f_y.

14. The bent cap and girders shall be capable of resisting unfactored dead load forces and moments combined with seismic forces and moments.

    The ultimate seismic moments to be considered shall be those that are the least critical of the following two cases:

    A. The moments which result from using the final top of column probable plastic moments as an applied load.

    B. The moments which result from an elastic seismic analysis before any reduction for ductility (Z factor). Two orthogonal directions of earthquake motion shall be considered.

       The moments which result from the analysis of earthquake motion in one direction shall be combined with 30 percent of the moments which result from the analysis of earthquake motion in the other direction. The two possible combinations of moments shall be considered.

    The ultimate seismic axial and shear forces to be considered shall be those associated with the least critical ultimate seismic moments.

    When determining the flexural capacity of the members, use a strength reduction factor (ϕ) equal to 1.0 and a yield strength of reinforcement equal to 1.0 times f_y.

    When determining the shear capacity of the members, use a strength reduction factor (ϕ) equal to 0.85 and a yield strength of reinforcement equal to 1.0 times f_y.

Columns on Combined (Common) Footing

1. Determine the column section requirements based on the Load Factor Design Group Loadings in AASHTO, and using the design strength of the member.

2. For each column determine the column probable plastic moments and the column axial forces and shear forces associated with the development of the probable plastic moments.

3. The ultimate moments to be used as applied moments for designing the footing shall be determined as specified in Step 5 of the procedures for "Columns on Individual Footings".

4. The ultimate horizontal forces and ultimate vertical forces to be used as applied forces for designing the footing shall be determined as specified in Steps 6 and 7 of the procedures for "Columns on Individual Footings".

5. In the transverse direction, assume the footing is a continuous beam on an elastic support, either soil or piles. In the longitudinal direction, assume the footing is a one-way footing. Using the soil or pile reactions obtained, design the footing sections. Consideration should be given to the tensile force applied to the footing due to the difference in column shears.

   Design a footing to resist the ultimate moments and forces of Steps 3 and 4. For resisting the vertical forces and moments use the ultimate soil bearing capacity or the ultimate pile bearing capacity and ultimate pile uplift capacity using a strength reduction factor (Ø) equal to 1.0. For resisting the lateral forces use the ultimate capacity of the soil or piles using a strength reduction factor (Ø) equal to 1.0.

   When determining the flexural capacity of the footing, use a strength reduction factor (Ø) equal to 1.0 and a yield strength of reinforcement equal to 1.0 times f_y. When determining the shear capacity of the footing, use a strength reduction factor (Ø) equal to 0.85 and a yield strength of reinforcement equal to 1.0 times f_y.

6. Design the piles of pile footings to sustain large curvatures and the design axial force. Refer to Step 9 of the procedures for "Columns on Individual Footings".

7. Check the footing design using the Load Factor Design Group Loadings in AASHTO, except omit Group VII. The ultimate soil bearing capacity shall be modified by a strength reduction factor (Ø) equal to 0.5 and the ultimate pile bearing capacity shall be modified by a strength reduction factor (Ø) equal to 0.75.

When checking the adequacy of the footing sections, use the design strength of the member specified in AASHTO. Revise footing, if required.

8. Design the column transverse reinforcement to provide for confinement and shear resistance. Refer to Step 11 of the procedures for "Columns and Individual Footings".

9. Design the connection of hinged columns to resist the ultimate forces determined in Step 4. Refer to Step 12 of the procedures for "Columns on Individual Footings".

10. The bent cap and girders shall be capable of resisting unfactored dead load forces and moments combined with seismic forces and moments. Refer to Step 14 of the procedures for "Columns on Individual Footings".

Columns as Extensions of Piles

1. Using the Load Factor Design Group Loadings in AASHTO, determine the required pile embedment and the required column and pile sections.

   For determining the required pile embedment to resist the applied moments and lateral forces use a limiting equilibrium analysis. For this analysis, use ultimate lateral soil pressures modified by a strength reduction factor ($\emptyset$) equal to 0.5. For determining the column and pile section requirements, use the design strength of the members.

   If the location of the maximum moment in the pile differs significantly from the location of pile fixity assumed for the frame analysis, consideration should be given to making a revised frame analysis.

2. Determine the column and pile probable plastic moments at the locations where plastic hinges may form. Assume the plastic hinge in the pile occurs at the point of maximum moment determined in Step 1, or if the column section probable plastic moment at the column/pile connection is less, assume a plastic hinge occurs at the column/pile connection.

   Determine the column and pile shear forces and axial forces that are associated with the development of the selected probable plastic moments. Reevaluate the probable plastic moments, shear forces, and axial forces for the effects of overturning, if necessary.

3. Using the ultimate moment at the location of the plastic hinge or point of maximum moments near the ground surface and the associated shear force as loads, check if the pile embedment determined in Step 1 is adequate. Use a limiting

equilibrium analysis using the ultimate lateral soil pressures modified by a strength reduction factor (Ø) equal to 1.0. Increase the pile embedment, if required.

The ultimate moment to be used shall be that which is the least critical of the following two cases:

A. The final column probable plastic moment.

B. The resultant moment from an elastic seismic analysis before any reduction for ductility (Z factor). Two orthogonal directions of earthquake motion shall be considered. The moments which result from the analysis of earthquake motion in one direction shall be combined with 30 percent of the moments which result from the analysis of earthquake motion in the other direction. The two possible resultant moments shall be considered.

4. Check the pile and superstructure deflections using the Service Load Group Loading in AASHTO except omit Group VII. Use the longest of the following pile embedment lengths:

   1) the length determined in Step 1.
   2) the length determined in Step 3.
   3) the length specified by the TransLab Engineering Geology to resist the axial forces.

   Use an elastic method of analysis, and revise the pile section and/or embedment, if required.

5. Design the column and pile transverse reinforcement to provide for confinement and shear resistance. Refer to Step 11 of the procedures for "Columns on Individual Footings".

6. Design the connection of columns hinged at the top of pile to resist the applied ultimate forces.

   The forces to be used shall be the least critical of the following two cases:

   A. The unfactored dead load forces combined with the forces associated with the development of the probable plastic moments at the top of the column.

   B. The unfactored dead load forces combined with the forces from an elastic seismic analysis before any reduction for ductility (Z factor). Two orthogonal directions of earthquake motion shall be considered. The forces which result from the analysis of earthquake motion in one direction shall be combined with 30 percent of the forces which result from the analysis of earthquake motion in the other direction.

The two possible sets of resultant forces shall be considered. Refer to Step 12 of the procedures for "Columns on Individual Footings".

7. The bent cap and girders shall be capable of resisting unfactored dead load forces and moments combined with seismic forces and moments. Refer to Step 14 of the procedures for "Columns on Individual Footings".

2.4 DESIGN OF ABUTMENTS

Two types of abutments are discussed, a seat-type and a diaphragm-type.

Design of Seat-Type Abutment

Procedures for the design of a seat-type abutment which uses piles at the end of the wingwalls plus the lever arm afforded by the wingwalls to resist overturning moments.

The procedure will be illustrated using an abutment for the 2-span box girder structure used in the concrete design course.

1. Determine the embedment of the diaphragm and the length of wingwalls required to satisfy site requirements.

2. Determine number and size of elastomeric bearing pads required using service loads from superstructure analysis. Refer to Memo to Designers 7-1.

3. Assume preliminary dimensions for wingwalls, abutment diaphragm, backwall, curtain wall, wingwall footing, and transverse shear key. See Figures A-2, A-3 and A-4.

4. Determine unfactored dead load of abutment, static earth pressure, seismic earth pressure, and horizontal and vertical forces transferred from the superstructure. See Figure A-5.

5. Using the load factors and group loadings in AASHTO, determine the number of piles required to resist the vertical and horizontal forces and overturning moments. Use the ultimate bearing capacity, ultimate lateral resistance and ultimate tensile capacity of the piles modified by a strength reduction factor (Ø) equal to 0.75 for all group loadings except VII for which use a strength reduction factor (Ø) equal to 1.0. For purposes of analysis assume a pinned support at the junction of abutment diaphragm and the front row of piles.

6. Using factored loads, design the following elements using the design strength of the member as specified in AASHTO:

1) abutment diaphragm
2) abutment backwall
3) longitudinal shear key at base of backwall
4) transverse shear key at end of abutment diaphragm
5) curtain wall
6) wingwalls
7) footing at end of wingwall.

In designing the abutment diaphragm consideration should be given to the torsional load created by the eccentricity of the earth pressure and forces transferred through the elastomeric bearing pads. This torque is resisted by the wingwalls and wingwall piles. The wingwalls should be designed for a vertical moment in the plane of the wingwalls to resist the torque from the abutment diaphragm.

In designing the wingwall, check the vertical shear capacity at the minimum section at the wingwall footing.

In designing the wingwall to backwall/abutment diaphragm connection, take into account the shear and moment due to earth pressure on the wingwall and the fact that for this type of corner detail, where the moment tends to open the corner, it is difficult to maintain the moment capacity of the wingwall section around the joint area. Consideration should be given to using a haunch in the corner even for abutments without skew.

Design of Diaphragm Abutment

Procedure for the design of a diaphragm abutment on either piles or continuous spread footing with cantilevered wingwalls.

The procedure is illustrated using an abutment for the 3-span T-beam structure used in the concrete design course.

1. Determine the dimensions of the diaphragm, wingwall, etc., to satisfy site requirements.

2. If diaphragm rests on elastomeric bearing pads, determine their size and number using service loads from superstructure analysis.

3. Determine factored axial load. Find number of piles or area of spread footing required using ultimate capacities of soil or piles reduced by appropriate Ø factor i.e. (0.5 for soil, 0.75 for piles).

4. Design abutment wall as cantilever using the design strength of the member as specified in AASHTO. Longitudinal force applied at base of cantilever will be dependent upon the type of support as follows:

On Concrete Piles

Longitudinal Force H_L = V(gross section) = $V_c + V_s$ (per pile)

$$V_s = A_v f_y d/s$$

$$V_c = 2(1 + .0005\ DL/A_g)\sqrt{f'_c}\, b_w d$$

Where: V_c = nominal shear strength provided by concrete.

V_s = nominal shear strength provided by shear reinforcement.

A_v = area of shear reinforcement within a distance s, in square inches.

f_y = specified yield strength of non-prestressed reinforcement, in psi.

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, but need not be less than 0.80h for prestressed members, in inches. (For circular sections, d need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member.)

s = spacing of shear reinforcement in direction parallel to longitudinal reinforcement, in inches.

DL = unfactored dead load.

A_g = gross area of section, in square inches.

f'_c = specified compressive strength of concrete, in psi.

b_w = web width, or diameter of circular section, in inches.

On Steel 45T Piles

Use $H_L = 30^k$ (per pile)

On Elastomeric Bearing Pads

Use H_L = 25% of unfactored dead load.

These values are to be used in lieu of more exact analysis which gives greater values. Minimum reinforcement to be #5 at 18" in both faces.

5. Wingwalls are designed using Group Loading and the following:

<u>Earth Pressure</u> $E = (0.5)\ K_a \gamma h^2$

Where: $K_a \gamma = 36$ pcf.

h = height of wall at point considered.

<u>Live Load</u> Regardless of Group used, consider as 2' equivalent earth surcharge.

For convenience the equations in BRIDGE DESIGN AIDS pages 3-6 may be used considering L of wall and s = 2' then applying factors:

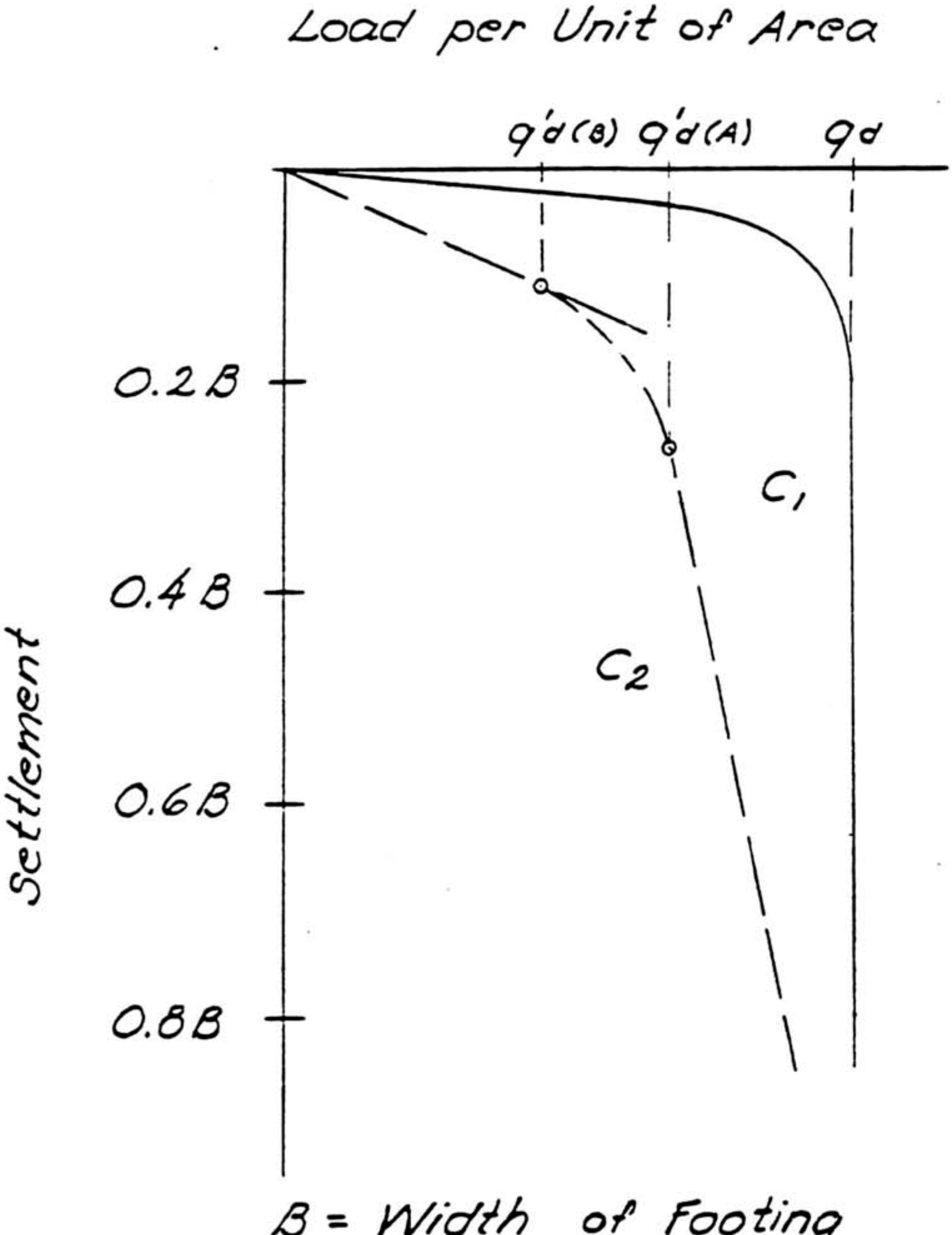
$$\gamma = 1.3$$

$$\beta = 1.3$$

Giving: $P_u = (1.69)\ (P)$

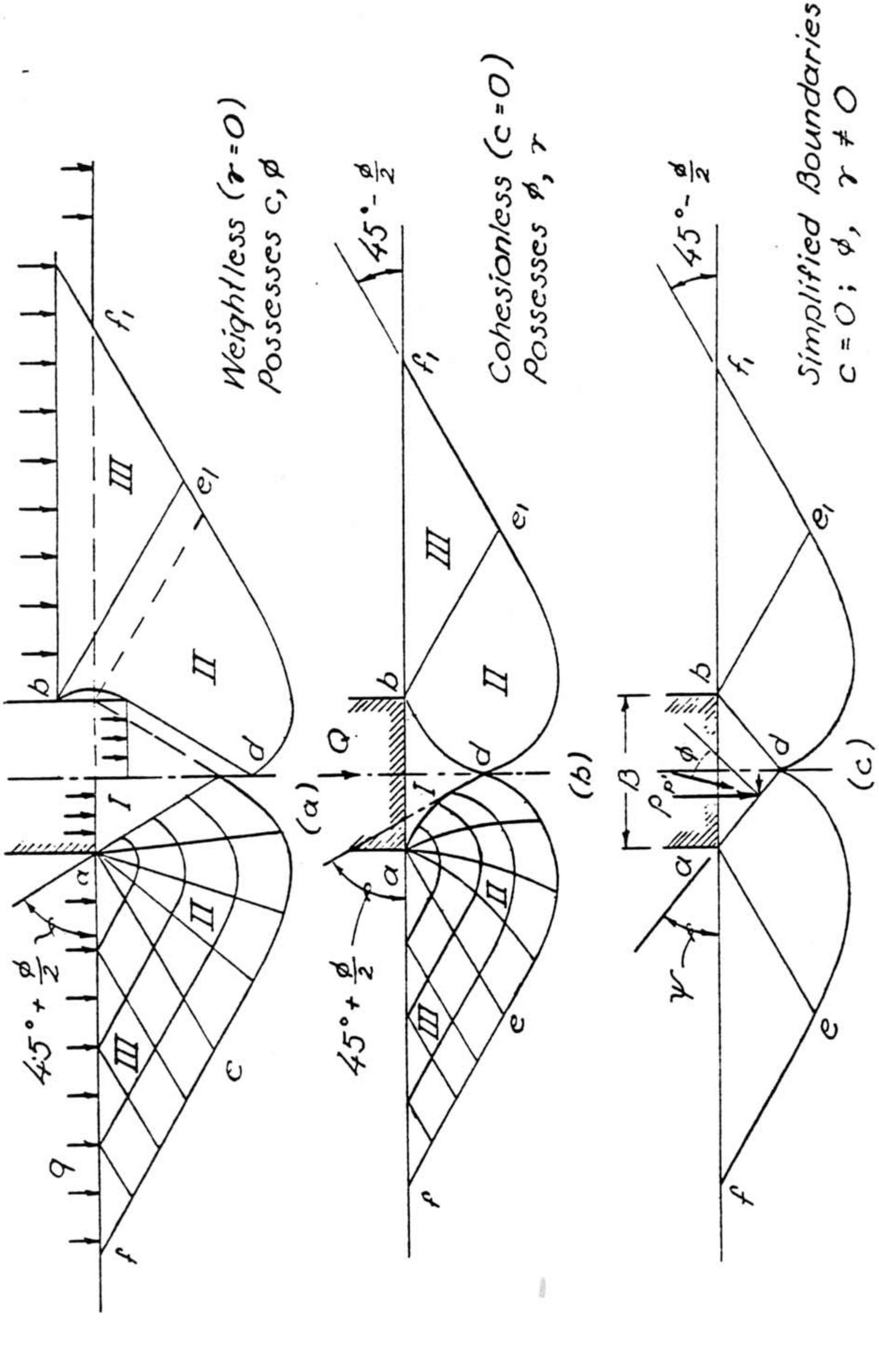
$$M_u = (1.69)\ (M_{AA})$$

BEARING CAPACITY OF SHALLOW FOOTINGS



Relation between intensity of load and settlement of a footing on C_1 dense or stiff and C_2 loose or soft soil.

Figure 2-1



Boundaries of zone of plastic equilibrium after failure of soil beneath continuous footing

Figure 2-2

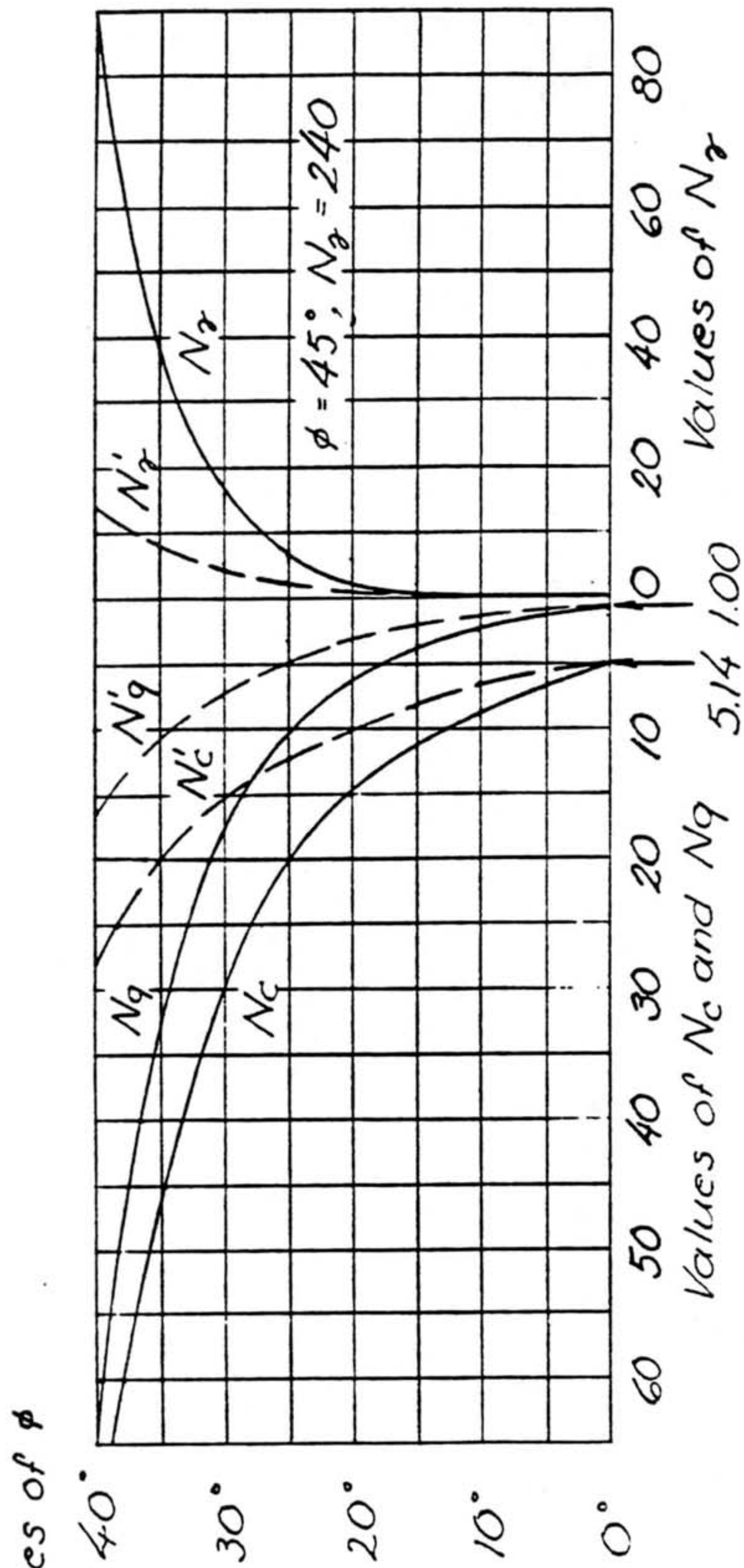


Chart showing relation between φ and bearing capacity factors (values of N_γ after Meyerhof 1955)

Figure 2-3

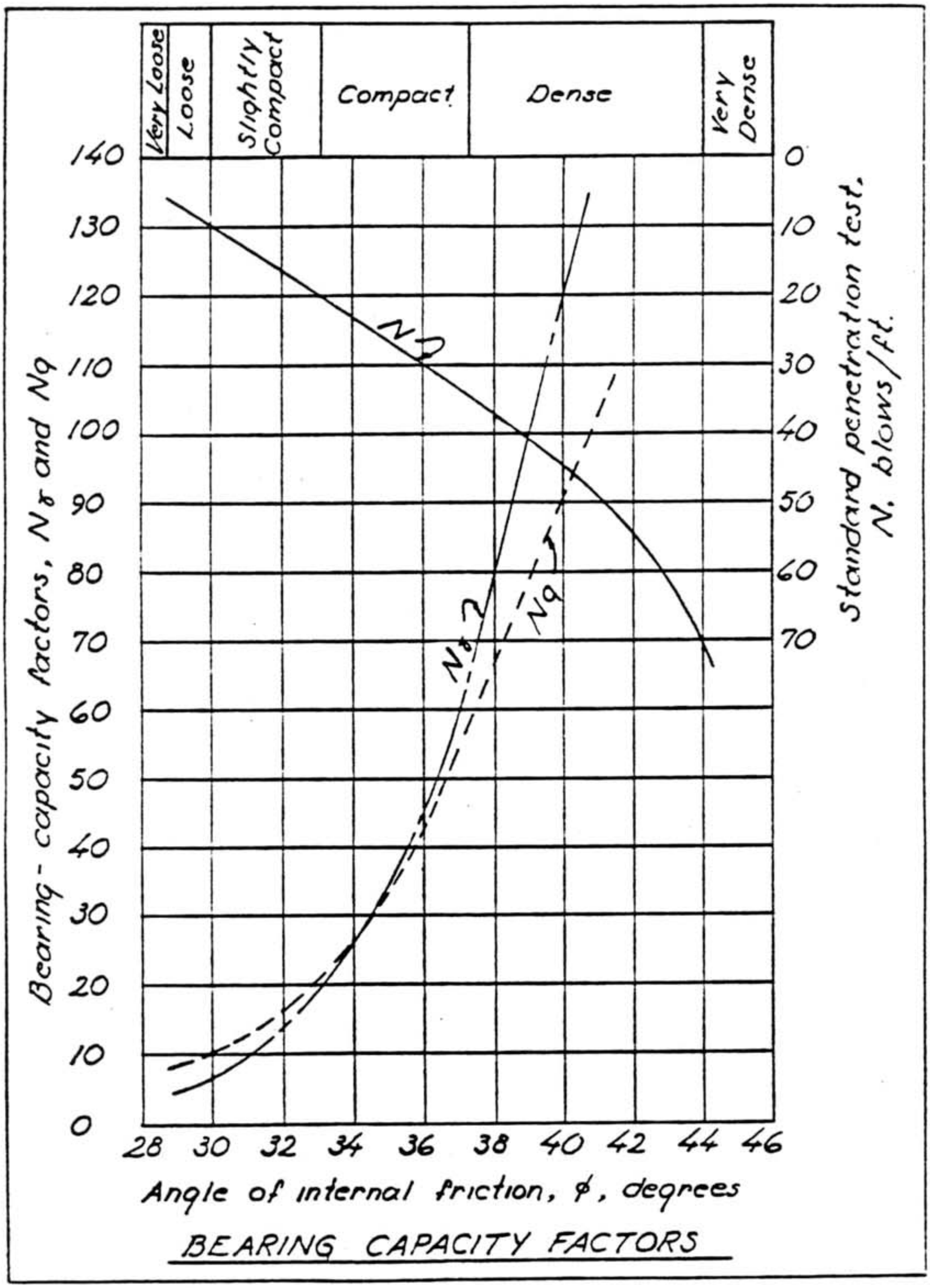

Very Loose
Loose
Slightly Compact
Compact
Dense
Very Dense
Bearing - capacity factors, Nγ and Nq
Standard penetration test, N, blows/ft.
Nγ
Nq
Angle of internal friction, φ, degrees
BEARING CAPACITY FACTORS

Figure 2-4

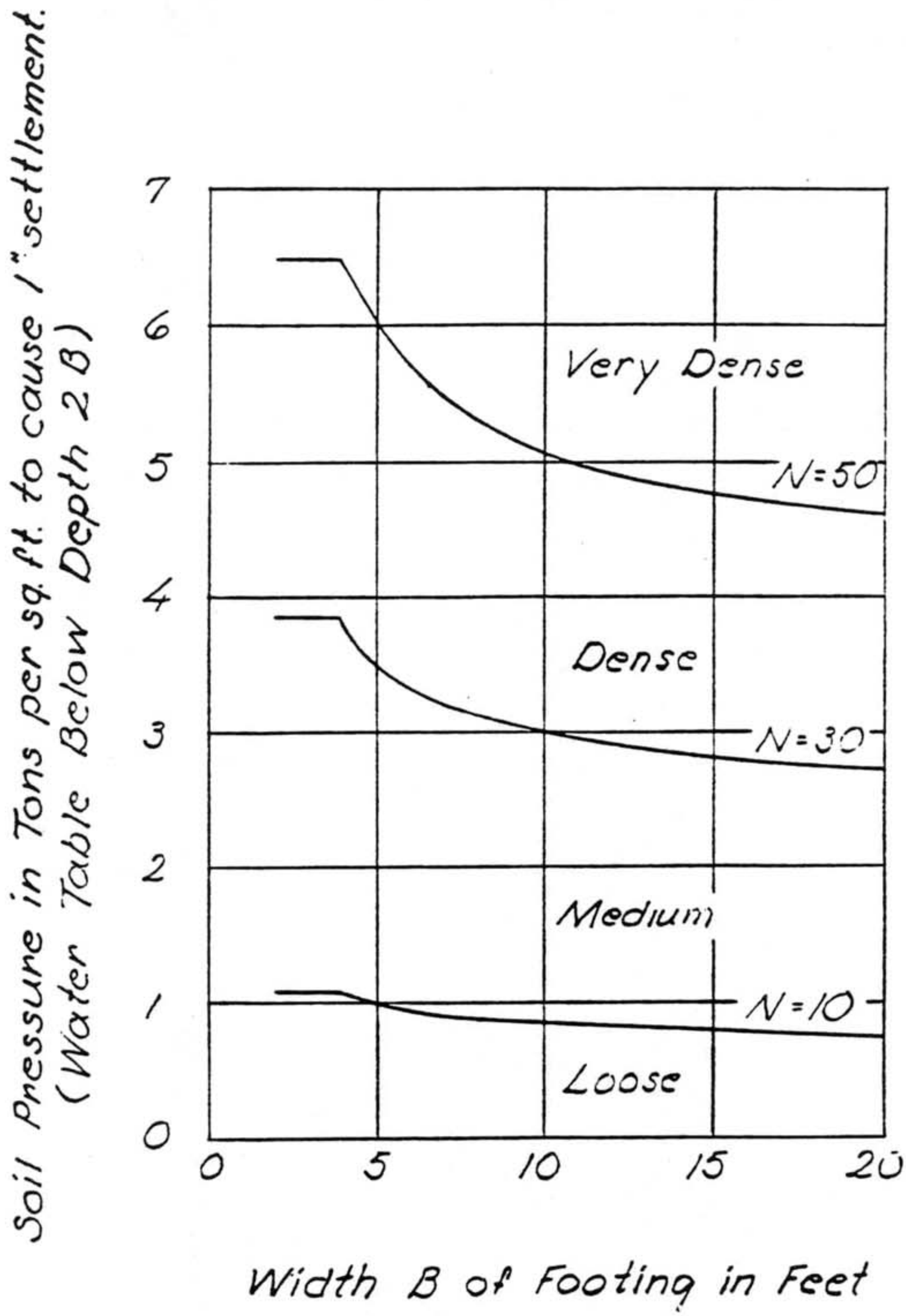


Chart for estimating allowable soil pressure for footing on sand on the basis of results of standard penetration test.

Figure 2-5

III. EARTH RETAINING STRUCTURES

3.1 STATES OF STRESS

When the maximum shearing strength is fully mobilized along a surface within a soil mass, a failure condition known as a state of plastic (or limiting) equilibrium is reached. Rankine's active and passive states of stress result when shear stresses equal to the maximum shearing strength of the soil develop uniformly and unhindered in two major directions throughout a soil mass due to lateral extension or compression.

Where the combinations of shear and normal stress with a soil mass all lie below the limiting envelope (see Fig. 3-1), the soil is in a state of elastic equilibrium. A special condition of elastic equilibrium is the "at-rest" state, where the soil is prevented from expanding or compressing laterally with changes in the vertical stress.

3.2 LIMITING EQUILIBRIUM CONDITIONS

The limiting equilibrium theories all require that the maximum shearing strength of the soil is mobilized. This however, requires deformation in the soil. The deformation of a supporting structure has only a local effect on the state of stress in the soil. The remainder of the soil remains in a state of elastic equilibrium. The state of stress in the locally disturbed zone and the shape of this zone is dependent on the amount and type of wall deformation. This also determines the shape of the pressure distribution on the wall and the intensity of the pressure. When a wall moves outward, the shearing strength of the retained soil resists the corresponding outward movement of the soil and reduces the earth pressures on the wall.

The earth pressure calculated for the <u>active</u> state is the absolute minimum value. When the wall movement is towards the retained soil, the shearing strength of the soil resists the corresponding soil movement and increases the earth pressure on the wall. The earth pressure (or resistance) calculated for the <u>passive</u> state is the maximum value that can be developed.

The amount of movement required to produce the active state in the soil is dependent mainly on the type of backfill material. Fig. 3-2 gives the outward movement of a wall which is necessary to produce an active state of stress in the retained soil. The movements required to produce full passive resistance are considerably larger, especially in cohesionless material. These requirements apply whether the movement is a lateral translation of the whole wall or a rotation about the base. The pressure distributions for full active and passive states are basically triangular for constantly sloping ground.

The amount of wall movement which will take place depends mainly upon the foundation conditions and the flexibility of the wall. The designer must insure that the calculated earth pressures correspond to the available wall movement. A free-standing wall need only be designed for active earth pressure as far as stability is concerned, since if it starts to slide or overturn under higher pressures, the movement will be sufficient to reduce the pressures to active. However, if it is on a strong foundation or otherwise fixed so that adequate stability is provided, the stem may be subject to pressures near those for the at-rest state.

3.2.1 The Rankine Earth Pressure Theory

Rankine's equations give the earth pressure on a vertical plane which is sometimes called the virtual back of the wall. The earth pressure on the vertical plane acts in a direction parallel to the ground surface and is directly proportional to the vertical distance below the ground surface (see Figure 3-3).

Provisions for Rankine's conditions in cohesive soils with a horizontal ground surface are available.

3.2.2 The Coulomb Earth Pressure Theory

The theory directly gives the resultant pressure against the back of a retaining structure for any slope of the wall and for a range of wall friction angles. It assumes that the soil slides on the back of the wall and mobilizes the shearing resistance between the back of the wall and soil as well as that on the failure surface.

The Coulomb equations reduce to those of the Rankine theory if a vertical wall surface with an angle of wall friction equal to the backfill slope is used. Other cases of wall slope or wall friction require curved surfaces of sliding to satisfy static equilibrium. The degree of curvature may be quite marked, especially for passive conditions. However, Coulomb's theory assumes that the failure wedge is always bounded by a plane surface, and it is therefore only an approximation for passive conditions. It is usually on the unsafe side if the wall friction angle exceeds 1/3 ϕ.

The simplifying assumption also means that static equilibrium is not always completely satisfied. For example, the forces acting on the soil wedge cannot all be resolved to act through a common point. The error from an exact solution is proportional to the amount by which static equilibrium is not satisfied.

In the active case the soil tends to slip downward along the back of the wall causing the resultant earth pressure to be inclined at a positive angle to the normal to the wall.

It is recommended that an angle of wall friction of + 2/3 of ϕ be used in the equation for active pressure for concrete walls which have been cast against formwork.

3.2.3 Passive Pressures Using Equations

The movements required to produce passive pressure lead to the soil sliding upward on the failure surfaces (including the back of the wall or anchor block). Therefore, Rankine's equation does not theoretically apply for passive resistance of soil with a positive ground slope against a vertical wall because it assumes a positive angle of wall friction equal to the ground slope, when in fact the wall friction angle would be negative. The use of Rankine's equation in this situation gives an underestimation of the passive resistance.

Equations for Coulomb's conditions allow the use of the correct direction and magnitude for the wall friction angle for passive pressure. However, for large positive backfill slopes or large values of wall friction, the error due to the assumption of a plane failure surface leads to a large overestimation of the passive resistance. This is accentuated further when the back of the wall has a negative slope. In the case of a vertical wall the Rankine equation should be used instead to give a conservative estimate of the passive resistance. For other wall slopes the passive resistance can be taken as Rankine's passive pressure on the vertical plane plus the weight of the soil wedge between the vertical plane and the pressure surface. Alternative methods based on curved failure surfaces, such as the logarithmic spiral method may be used. (See Reference 6)

For most cases involving passive pressures encountered in retaining wall design, the ground surface is horizontal and the pressure surface may be assumed to be vertical. If the angle of wall friction is taken as zero under these conditions, the Rankine and Coulomb equations are the same and the resulting passive resistance is on the conservative side. (Since there would be some wall friction which increases the passive resistance.)

3.2.4 The Trial Wedge Method

Where the ground surface is irregular, or where it is constantly sloping in cohesive soil, a graphical procedure using the assumption of planar failure surfaces is the simplest approach. This procedure is known as the "Trial Wedge Method".

The backfill is divided into wedges by selecting planes through the heel of the wall. The forces acting on each of these wedges are combined in a force polygon so that the magnitude of the resultant earth pressure can be obtained. A force polygon is constructed even though the forces acting on the wedge are often not in moment equilibrium. This method is therefore an approximation with the same assumption as the equations for Coulomb's conditions, and for a ground surface with a constant slope will give the same result. If the conditions are the same as those for Rankine's equations the Trial Wedge earth pressures will correspond to these also. The limitations on wall friction and passive pressures mentioned in the use of the Rankine and Coulomb equations also apply to the Trial Wedge Method. The adhesion of the soil to the back of the wall in cohesive soils is neglected since it increases the tension crack depth and hence reduces the active pressure.

For the active case the maximum value of the earth pressure calculated for the various wedges is required. This is obtained by interpolating between the calculated values. For the passive case the required minimum value is similarly obtained.

The direction of the resultant earth pressure and the force polygons should be obtained from the consideration of Sections 3.2.1 to 3.2.3. For the cases where this force acts parallel to the ground surfaces, a substitute constant slope should be used, as shown on Figure 3-4, for soil both with and without cohesion.

For an irregular ground surface the pressure distribution is not triangular. However, if the ground does not depart significantly from a plane surface, a linear pressure distribution may be assumed and the constructions given in Figure 3-5 and 3-6 used. A more accurate method is given in Figure 3-7. The latter should be used when there are nonuniform surcharges.

3.3 ELASTIC EQUILIBRIUM CONDITIONS

(At-Rest Pressures)

The special state of elastic equilibrium known as the at-rest state is useful as a reference point for calculation of earth pressures where only small wall movements occur. For the case of a vertical wall and a horizontal ground surface the coefficient of at-rest earth pressure may be taken as:
$K_o = 1 - \sin \phi'$ for normally consolidated materials. This assumes that the material has no built in overconsolidation stress. For other wall angles and backfill slopes, it may be assumed the K_o varies proportionally to K_A. At-rest earth pressures may be assumed to increase linearly with depth from zero at the ground surface for all materials.

The total at-rest earth pressure force is given by:

$$P_o = 1/2\ K_o \gamma H^2$$

This acts as H/3 from the base of the wall (or bottom of the key for walls with keys).

For gravity type retaining walls the at-rest pressure should be taken as acting normal to the back of the wall (i.e. $\delta = 0$). For cantilever and counterfort walls it should be calculated on the vertical plane through the rear of the heel and taken as acting parallel with the ground surface.

In cohesionless soils, full at-rest pressures will occur only with the most rigidly supported walls. In highly plastic clays, pressures approaching at-rest may devlop unless wall movement can continue with time (creep).

3.4 SEISMIC EARTH PRESSURE

The most frequently used method for the calculation of the seismic soil forces acting on bridge abutments or retaining walls is the static approach developed by Mononobe and Okabe. The Mononobe-Okabe analysis is an extension of the sliding-wedge theory taking into account horizontal and vertical inertia forces acting on the soil. The analysis is described in detail by Seed and Whitman, (Reference 4). The following assumptions are made:

1. The abutment is free to move sufficiently so that the soil strength will be mobilized. If the abutment is rigidly fixed and unable to move, the soil forces will be very much higher than those predicted by the Mononobe-Okabe analysis.

2. The backfill is cohesionless with a friction angle $\emptyset$.

3. The backfill is unsaturated, so that liquification problems will not arise.

3.5 SURCHARGES

Uniform Loads

Uniform surcharge loads may be converted to an equivalent height of fill and the earth pressures calculated for the correspondingly greater height.

Line Loads

Where there is a superimposed line load running a considerable length parallel to the wall, the weight per unit length of this load can be added to the weight of the particular trial wedge to which it is applied (Fig. 3-8). The increased total earth pressure will be given from the trial wedge procedure but the line load will also change the point of application of this total pressure. The method given in Figure 3-7 may be used to give the distribution of pressure.

When the line load is small in comparison with active earth pressure, the effect of the line load on its own should be determined by a method based on stresses in an elastic medium. The pressures thus determined are superimposed on those due to active earth pressure and other effects (Ref. 2, Sheet 7-10-10).

Point Loads

Point loads cannot be taken into account by trial wedge procedures. The method based on Boussinesq's equations should be used (Ref. 2, Sheet 7-10-10).

Static Water Level

Where part or all of the soil behind the wall is submerged below a static water level, the earth pressure is changed due to the hydrostatic pore pressures set up in the soil. The water itself also exerts lateral pressure on the wall equal to the depth below the water table times the density of water.

3.6 STABILITY OF RETAINING WALLS

3.6.1 General

The stability of a freestanding retaining structure and the soil containing it is determined by computing the factors of safety or "stability factors" which may be defined in general terms as:

$$F.S. = \frac{\text{moments or forces aiding stability}}{\text{moments or forces causing instability}}.$$

Factors of safety should be calculated for the following separate modes of failure:

a) Sliding of the wall outwards from the retained soil.
b) Overturning of the retaining wall about its toe.
c) Foundation bearing failure.
d) Slip circle failure in the surrounding soil.

When calculating overall stability of the wall, the lateral earth pressure is calculated to the bottom of the footing, or in the case of a footing with a key, to the bottom of the key.

The vertical component (if any) of the resultant earth pressure may be added to the weight of the wall system when computing stability factors.

If the passive resistance of the soil in front of the wall is included in calculations for stability, the top 12" of the soil should be neglected, and passive resistance should be calculated by Rankine theory.

3.6.2 Sliding Stability

Factor of safety:

$$F.S. = \frac{\text{sum of the forces resisting sliding}}{\text{sum of the forces causing sliding}}$$

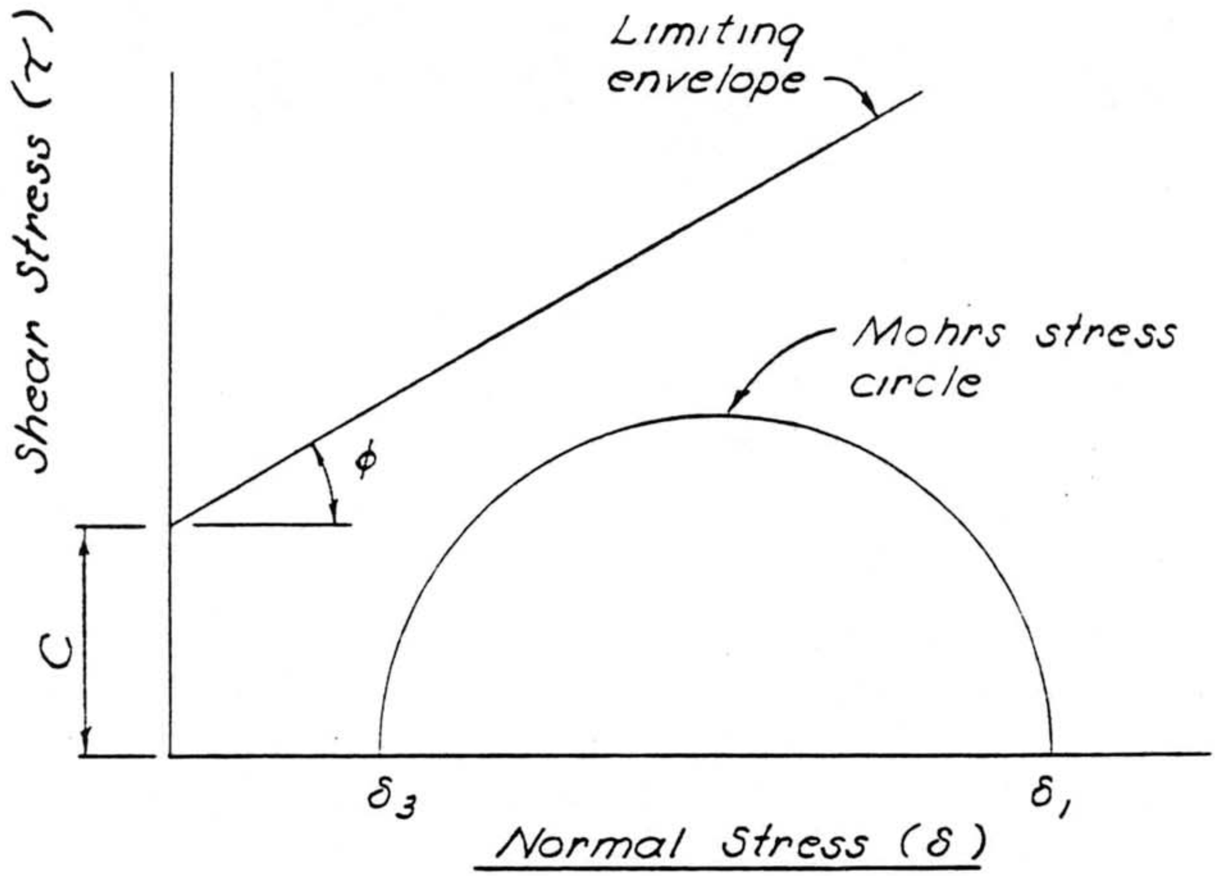
should be at least equal to 1.5 for static loading and at least 1.2 for seismic loading.

3.6.3 Overturning Stability

Moments calculated about the bottom of the front of the toe must give an overturning factor of safety:

$$F.S. = \frac{\text{sum of the moments resisting overturning}}{\text{sum of the moments causing overturning}}$$

The factor of safety for overturning should be at least 2.0 for static loading. For seismic loading F.S. for sliding is generally exceeded before overturning is critical.



Stresses in soil in elastic range - (below limiting envelope)

Figure 3-1

SOIL	WALL YIELD
Cohesionless, dense	0.001 H
Cohesionless, loose	0.001 - 0.002 H
Clay, firm	0.01 - 0.02 H
Clay, soft	0.02 - 0.05 H

H = Height of wall

Movement of wall necessary to produce active pressures.

Figure 3-2

NOTES

1. Material shaded ////////// is included in the total weight for calculation of sliding stability.

2. Adequate drainage is assumed - otherwise lateral hydrostatic water pressure would have to be included.

3. The earth pressure denoted by * is used for the stem design.

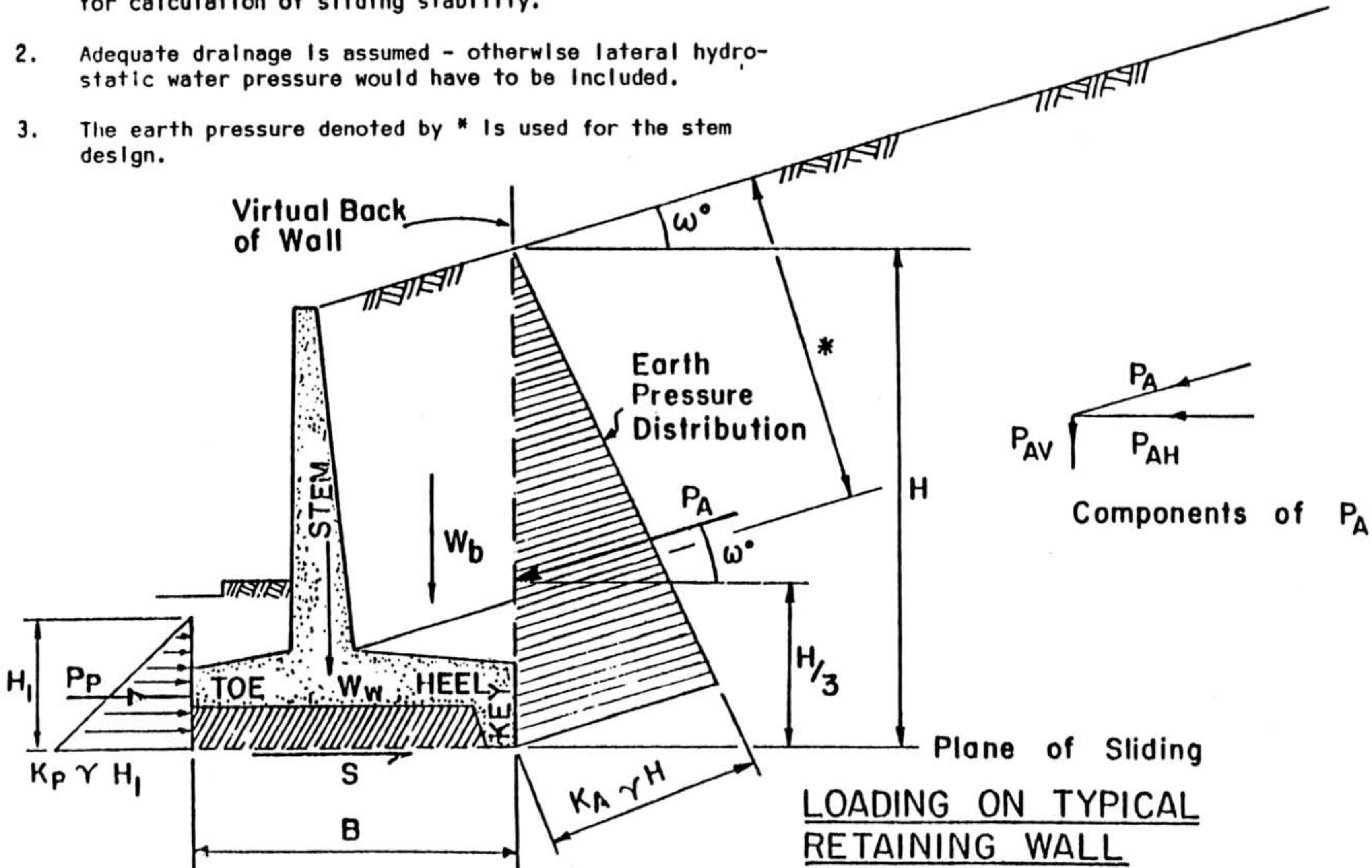
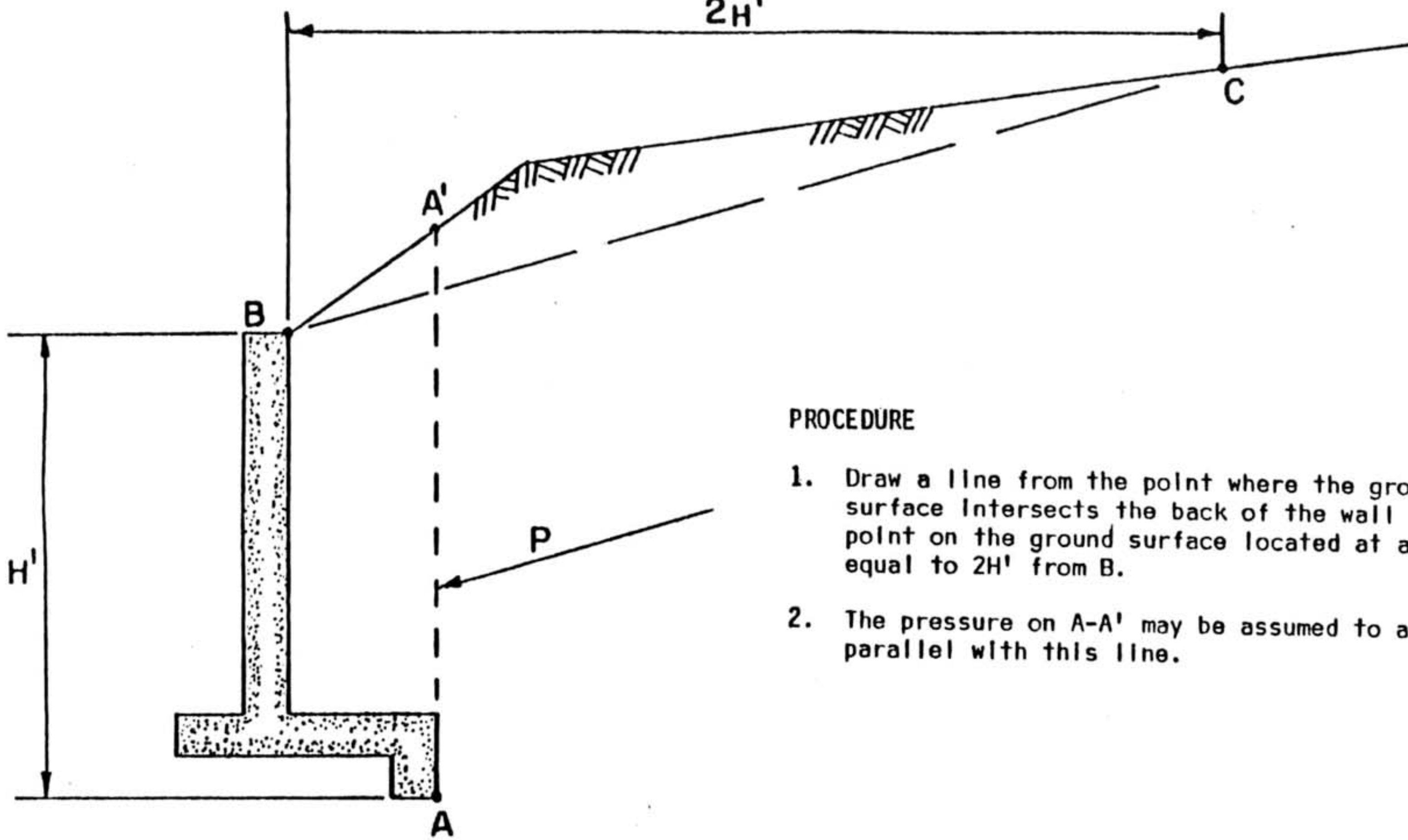


Figure 3-3



PROCEDURE

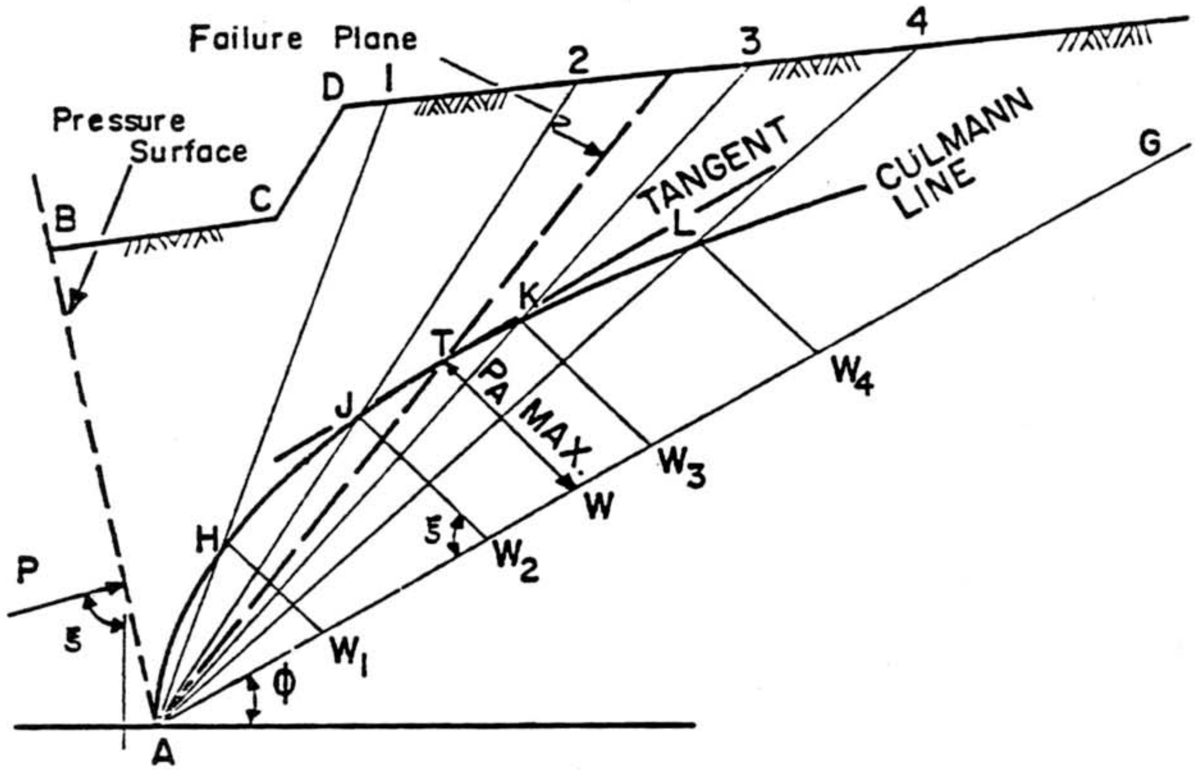
1. Draw a line from the point where the ground surface intersects the back of the wall (B) to a point on the ground surface located at a distance equal to 2H' from B.

2. The pressure on A-A' may be assumed to act parallel with this line.

APPROXIMATE METHOD FOR DIRECTION OF RANKINE EARTH PRESSURE

Figure 3-4

TRIAL WEDGE METHOD
COHESIONLESS SOIL
CULMANN'S CONSTRUCTION
(FOR STATIC EARTH PRESSURE ONLY)

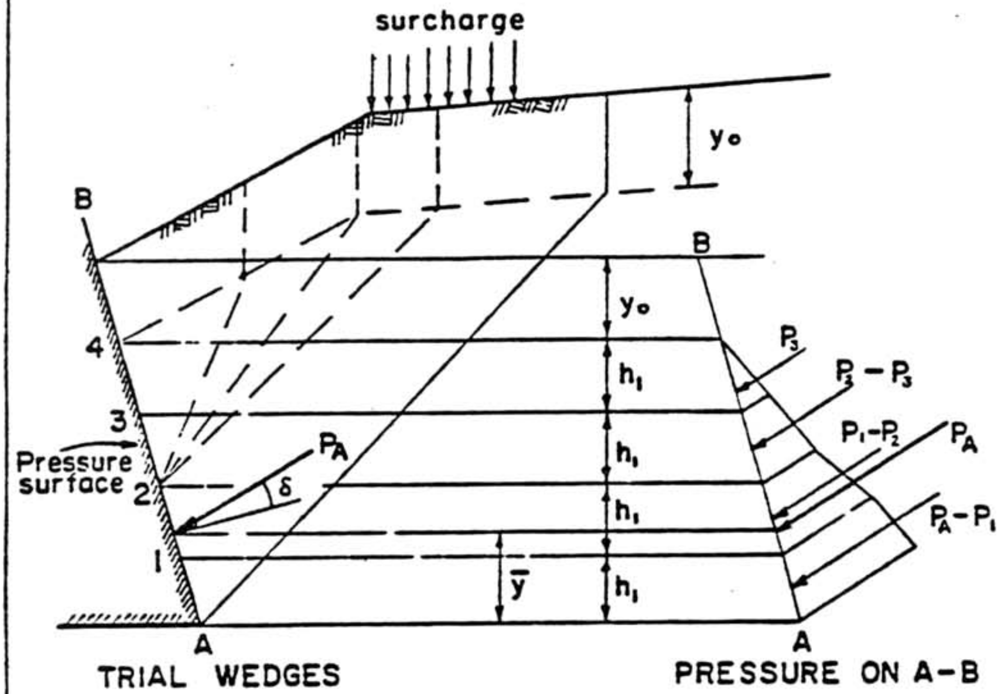


PROCEDURE

1. Draw line A-G at an angle of ϕ^o to the horizontal for active pressure.
2. Draw trial wedges ABCD1, ABCD2, etc. - a minimum of four will usually suffice.
3. Calculate the weights of the wedges - say w_1, w_2, etc., and plot these to a suitable scale on A-G, each measured from A.
4. Through w_1, w_2, etc., draw lines at an angle ξ, (see text for direction of P_A and hence ξ), to intersect A-1, A-2, etc., at H, J, etc.
5. Draw a curve through A, H, J, etc.
6. P_A is obtained by drawing a tangent to the curve, parallel to A-G to touch at T. P_A is the line W-T, to the same scale as w_1, etc.
7. The failure plane is the line through A and T.

Figure 3-5

POINT OF APPLICATION OF RESULTANT PRESSURE AND PRESSURE DISTRIBUTION



Use when the ground surface is very irregular or when a non-uniform surcharge is carried.

PROCEDURE

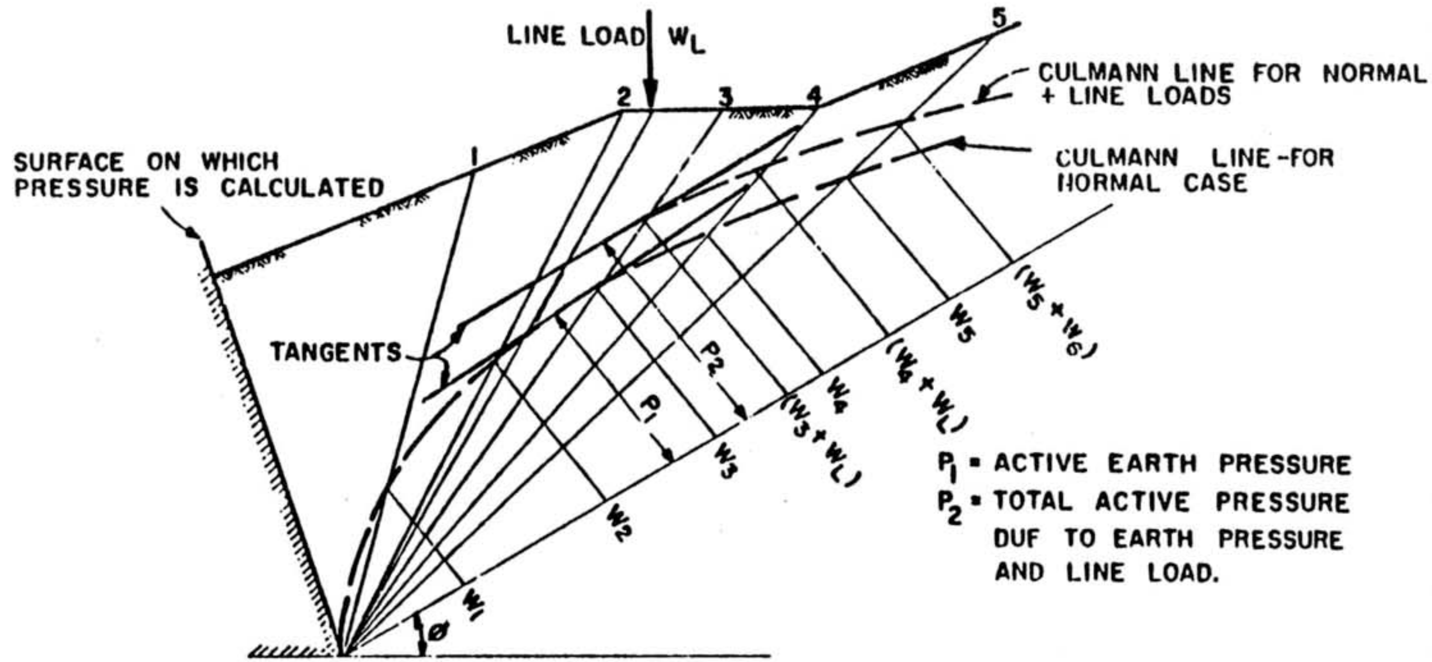
1. Subdivide the line A-4 into about 4 equal parts h_1 (below the depth y_o of tension cracking).

2. Compute the active earth pressures P_1, P_2, P_3, etc., as if each of the points 1, 2, 3, etc., were the base of the wall. The trial wedge method is used for each computation.

3. Determine the pressure distribution by working down from point 4. A linear variation of pressure may be assumed between the points where pressure has been calculated.

4. Determine the elevation of the centroid of the pressure diagram, $\bar{y}$. This is the approximate elevation of the point of application of the resultant earth pressure, P_A.

Figure 3-7



ANALYSIS SAME AS FOR STANDARD CASE, UNTIL TRIAL WEDGE INCLUDES LINE LOAD. THE LINE LOAD ADDED TO THE INDIVIDUAL TRIAL WEDGE' WEIGHTS INCREASES THE VALUE OF P_A.

TRIAL WEDGE OF ACTIVE PRESSURE INCLUDING LINE LOAD

Figure 3-8

SUBSTRUCTURES AND FOUNDATIONS

IV. REFERENCES

1. "Soil Mechanics in Engineering Practice, 2nd Edition," by Terzaghi, K. and Peck, R. B. (1967) John Wiley & Sons.

2. "Design Manual Soil Mechanics, Foundations, and Earth Structures," NAVFAC DM-7, U.S. Department of the Navy (1971).

3. "Foundation Design," by Teng, Wayne C. (1962), Prentice-Hall Inc.

4. "Design of Earth Retaining Structures for Dynamic Loads," Seed, H. B. and Whitman, R. V. (1970), ASCE Specialty Conference - Lateral Stresses in the Ground and Earth Retaining Structures.

5. "Retaining Wall Design Notes," New Zealand Ministry of Works (1973) Design Manual prepared in the Office of the Chief Design Engineer (civil).

6. "Steel Sheet Piling Design Manual," U.S. Steel Corp. (1975).

7. "Trenching and Shoring Manual," State of California Department of Transportation (1977).

8. "Handbook of Engineering Geology," State of California Department of Transportation (1958).

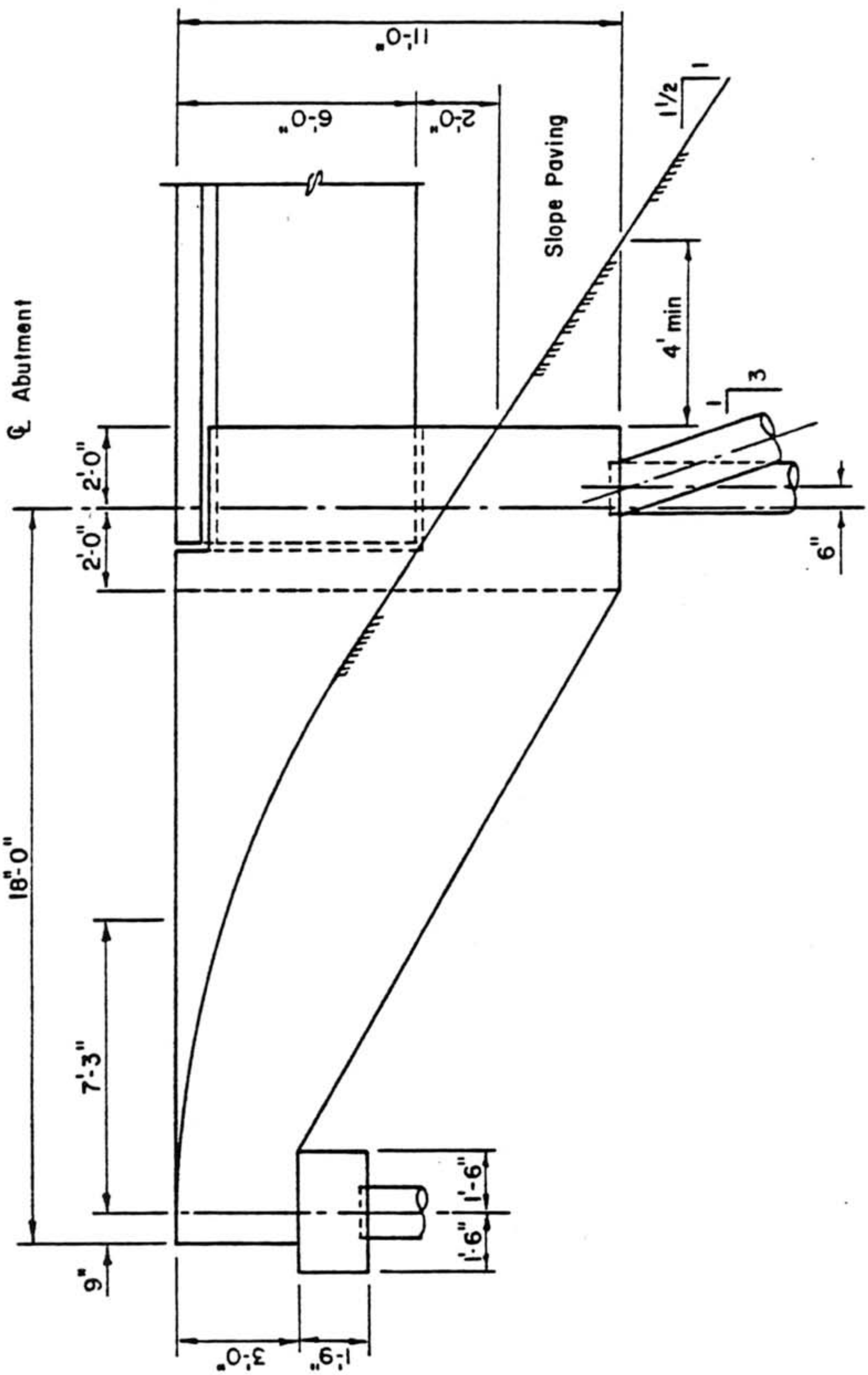
9. ATC-6, draft copy dated March 7, 1979 by the Applied Technology Council.

10. "Recommended Lateral Force Requirements and Commentary," (1975) by Structural Engineers Association of California (SEAOC).

11. "Standard Specifications for Highway Bridges, Twelfth Edition," (1977) by the American Association of State Highway and Transportation Officials (AASHTO).

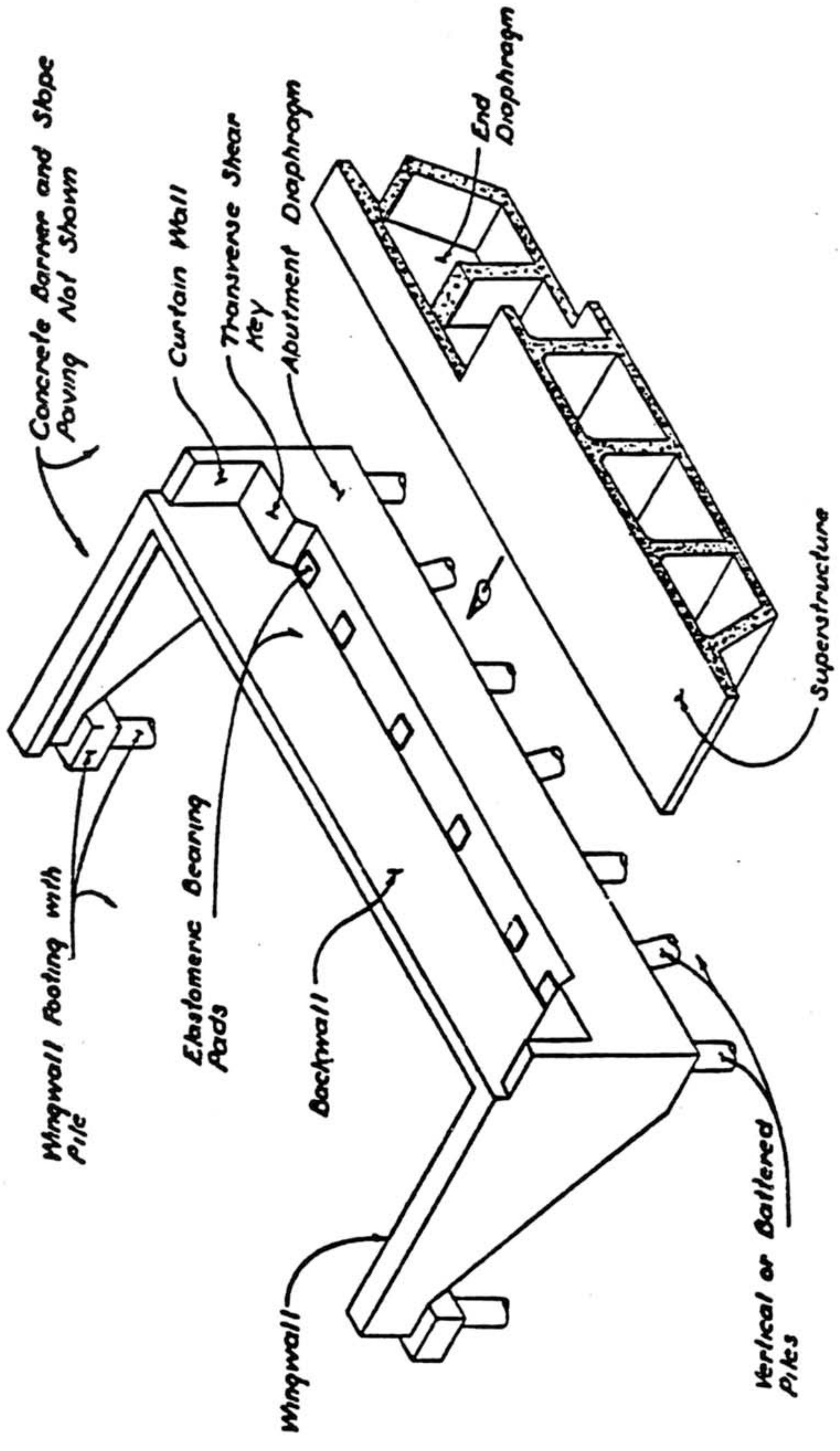
12. "Bridge, Memos to Designers," State of California Department of Transportation

13. "Standard Plans," State of California Department of Transportation (March 1977).



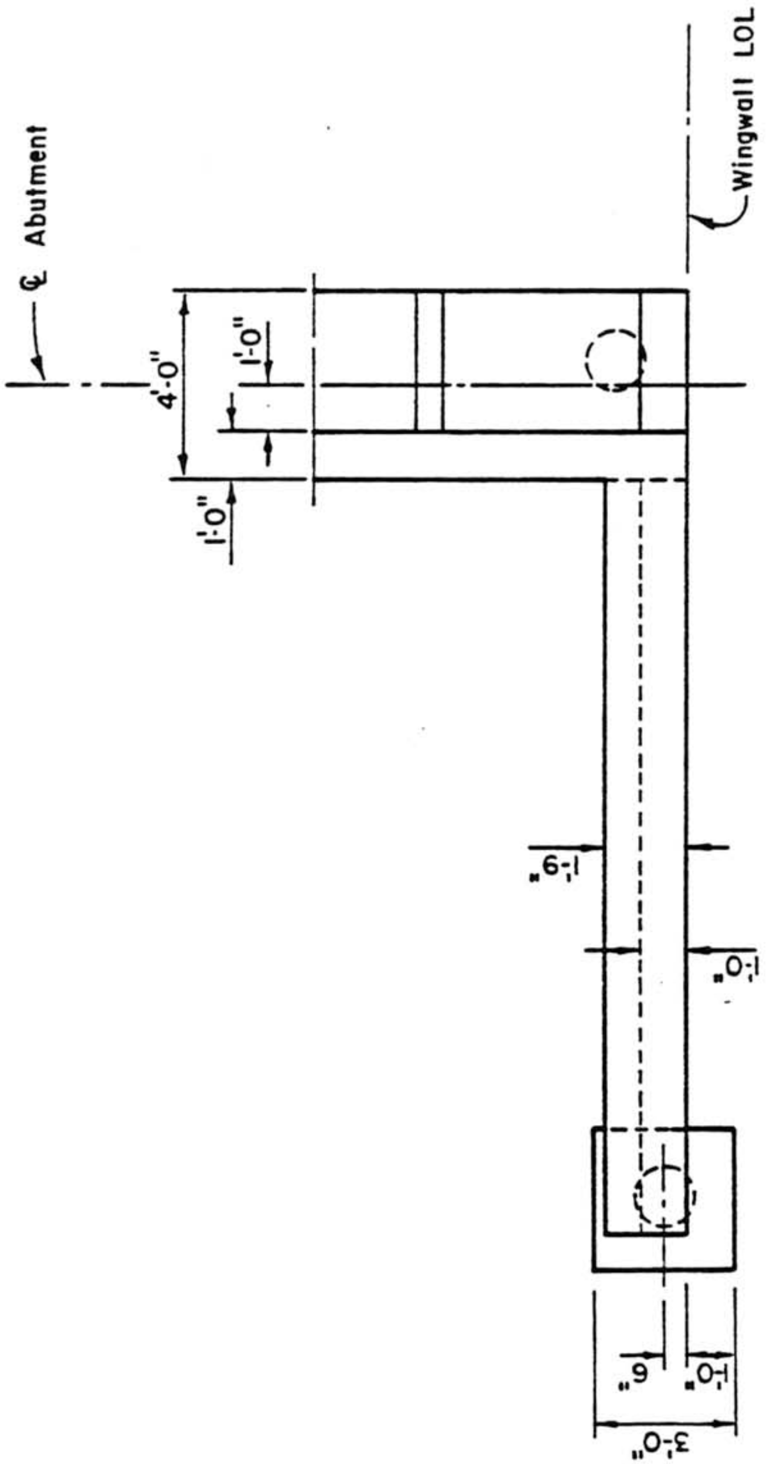
SIDE ELEVATION
1" = 4'

Figure A-1



ISOMETRIC VIEW

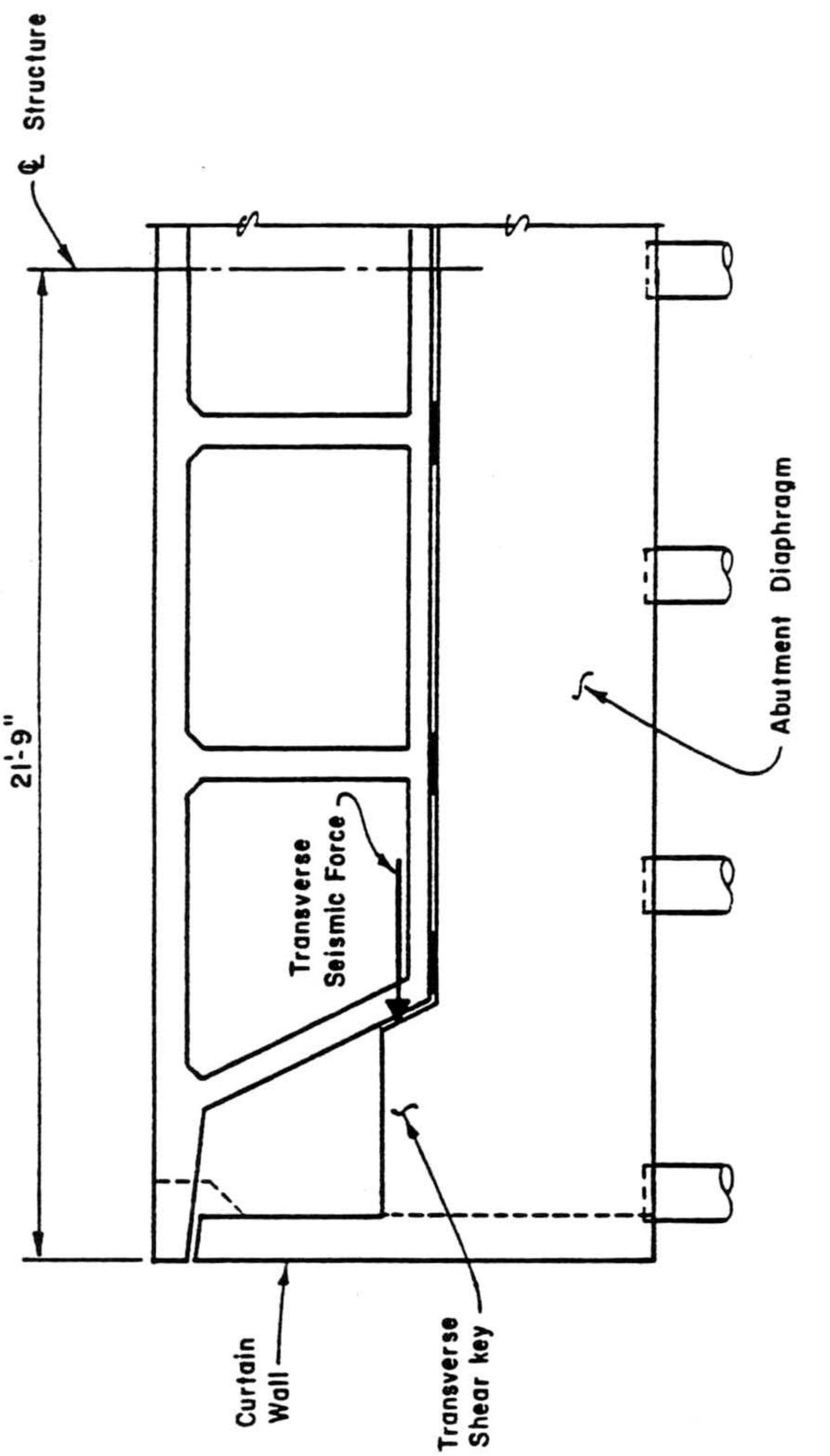
Figure A-2



PLAN WINGWALL

1" = 4'

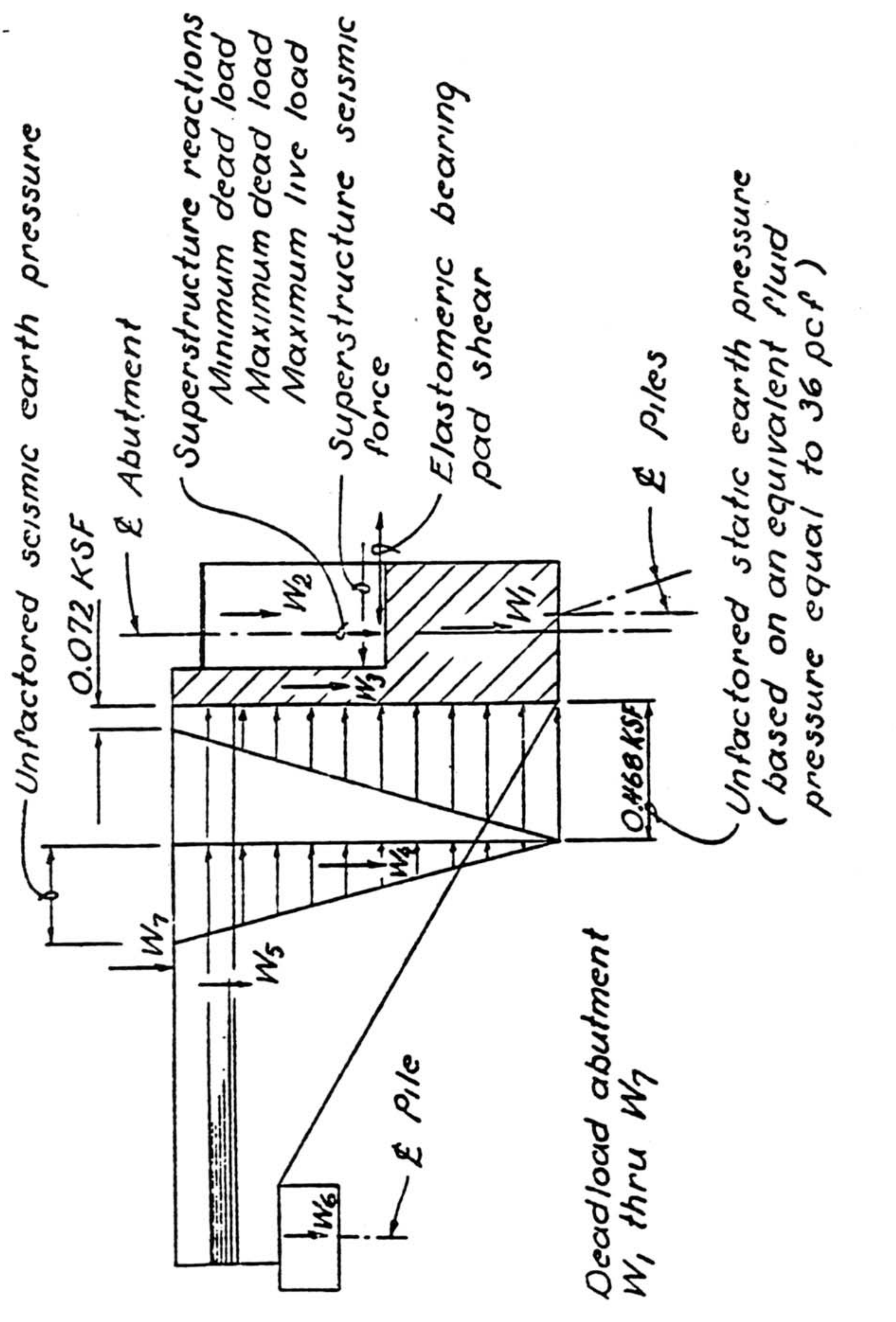
Figure A-3



PART FRONT ELEVATION

1" = 4'

Figure A-4



SECTION

Figure A-5

Example Problem 1 - Single Column Bent W/Pile Footing

Problem: Design the column and a pile footing for the single column bents of the 3 span box girder bridge used in the Reinforced Concrete Section of the Bridge Design Practice Manual.

Note: Article numbers cited within this example problem refer to the article numbers used in the AASHTO Standard Specifications for Highway Bridges, 12th Edition and including Interim Specifications through 1981 and Caltrans Bridge Design Specifications.

Column Loads: For illustrative purposes only dead load, live load and seismic loads will be considered. The X-axis equals the centerline of bent and the Y-axis equals the centerline of column.

Dead load (service level)

$P^{DL}_{TOP\ COL} = 1056^k$ $\qquad P^{DL}_{BOT\ COL} = 1154^k$

$M_x{}^{DL}_{TOP\ COL} = -524$ ft-kips $\qquad M_x{}^{DL}_{BOT\ COL} = 216$ ft-kips

$M_y{}^{DL}_{TOP\ COL} = 0$ $\qquad M_y{}^{DL}_{BOT\ COL} = 0$

Live load + impact (service level); Impact = 22%

Case 1

$P^{LL+I}_{TOP\ COL} = 224^k$ $\qquad P^{LL+I}_{BOT\ COL} = 224^k$

$M_x{}^{LL+I}_{TOP\ COL} = -146$ ft-kips $\qquad M_x{}^{LL+I}_{BOT\ COL} = 60$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 1121$ ft-kips $\qquad M_y{}^{LL+I}_{BOT\ COL} = 1121$ ft-kips

Case 2

$P^{LL+I}_{TOP\ COL} = 131^k$ $\qquad P^{LL+I}_{BOT\ COL} = 131^k$

$M_x{}^{LL+I}_{TOP\ COL} = -1252$ ft-kips $\qquad M_x{}^{LL+I}_{BOT\ COL} = 516$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 654$ ft-kips $\qquad M_y{}^{LL+I}_{BOT\ COL} = 654$ ft-kips

Live load + impact (for factored level); Impact = 22%

Case 1 (1.15 x 1 lane P + 1 lane H)

$P^{LL+I}_{TOP\ COL} = 456^{k}$ $P^{LL+I}_{BOT\ COL} = 456^{k}$

$M_x{}^{LL+I}_{TOP\ COL} = -330$ ft-kips $M_x{}^{LL+I}_{BOT\ COL} = 136$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 3672$ ft-kips $M_y{}^{LL+I}_{BOT\ COL} = 3672$ ft-kips

Case 2 (1.15 x 1 lane P + 1 lane H)

$P^{LL+I}_{TOP\ COL} = 269^{k}$ $P^{LL+I}_{BOT\ COL} = 269^{k}$

$M_x{}^{LL+I}_{TOP\ COL} = -2311$ ft-kips $M_x{}^{LL+I}_{BOT\ COL} = 952$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 2179$ ft-kips $M_y{}^{LL+I}_{BOT\ COL} = 2179$ ft-kips

Case 3 (1.15 x 1 lane P)

$P^{LL+I}_{TOP\ COL} = 204^{k}$ $P^{LL+I}_{BOT\ COL} = 204^{k}$

$M_x{}^{LL+I}_{TOP\ COL} = -1685$ ft-kips $M_x{}^{LL+I}_{BOT\ COL} = 694$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 2244$ ft-kips $M_y{}^{LL+I}_{BOT\ COL} = 2244$ ft-kips

Seismic load (ARS forces and moments before application of $Z = 6$ factor)

$H_x{}^{EQ}_{ARS} = H_y{}^{EQ}_{ARS} = 828^k$ (Horizontal force)

Case 1

$P^{EQ}_{TOP} = \pm 55^k$

$M_x{}^{EQ}_{TOP\ COL} = 8472$ ft-kips

Case 2 ($1.0\ EQ_L + 0.33\ EQ_T$)

$P^{EQ}_{BOT} = \mp 55^k$

$M_x{}^{EQ}_{BOT\ COL} = \pm 7985$ ft-kips

$M_y{}^{EQ}_{BOT\ COL} = \pm 12624$ ft-kips

Case 3 ($0.33\ EQ_L + 1.00\ EQ_T$)

$P^{EQ}_{BOT} = \mp 18^k$

$M_x{}^{EQ}_{BOT\ COL} = \pm 2640$ ft-kips

$M_y{}^{EQ}_{BOT\ COL} = \pm 38256$ ft-kips

Column:

Geometry - Standard architectural column type 2R
Clear height = 20'-0"
Length of top flare section = 16'-6"
t= 5'-6"

Longitudinal reinforcement - determined by using YIELD program with reinforcement placement controlled by basic section of column.

$A_{s\ TOP}$ = 36 - #9 = 36.00 sq.in.

$A_{s\ BOT}$ = 54 - #9 = 54.00 sq.in.

9 - #9 extend 11' above top of footing

9 - #9 extend 8' above top of footing

A_g TOP COL ACTUAL = 7777.19 sq.in.

ρ_s = 36.0/7777.19 = 0.0046

A_g TOP COL DESIGN = 3421.19 sq.in.

ρ_s = 36.0/3421.19 = 0.0105 > 0.01 ok

A_g BOT COL ACTUAL & DESIGN = 3421.19 sq.in.

ρ_s = 54.0/3421.19 = 0.0158 > 0.01 ok

Article 1.5.11(A)(2) provisions were used for complying with the minimum ρ_s requirements of Article 1.5.11(A)(1).

At the base of the column the ρ_s = 0.0158 more than satisfies the minimum ρ_s = 0.005 specified in Article 1.4.6(J)(4)(C).

Nominal moment strength - Article 1.5.2(B)

f'_c = 3250 psi

f_y = 60000 psi

ϵ_c max = 0.003 in./in.

E_s = 29000000 psi

- Using the output from the YIELD program determine the nominal moment strength at sections within the flared portion of the column associated with the dead load plus seismic axial load.

Top of column

$Pe = 1056 + 55 = 1111^k$

$M_{nx} = 8190$ ft-kips

$M_{ny} = 14721$ ft-kips

5'-6" from top of column

$Pe = 1094 + 38 = 1132^k$

$M_{nx} = 7704$ ft-kips

$M_{ny} = 9752$ ft-kips

11'-0" from top of column

$Pe = 1121 + 26 = 1147^k$

$M_{nx} = 7075$ ft-kips

$M_{ny} = 7437$ ft-kips

Probable plastic moment - Article 1.5.33(D)

Basic section @ top of column

$Pe = 1111^k$

$M_{px} = M_{py} = 8233$ ft-kips

Basic section @ bottom of column

$Pe = 1154 + 55 = 1209^k$

$M_{px} = M_{py} = 10641$ ft-kips

Column shear - Article 1.5.35(G)(1)

Determine the maximum column shears considering that the nominal moment strengths can be developed in the gross flare sections and that the probable plastic moment strengths will be developed in the basic section.

Case 1 (nominal moment @ top of column, probable plastic moment @ bottom of column)

$$V_{ux} = \frac{(0 + 10641)}{(20.0 + 3.24)} = 458^k$$

$$V_{uy} = \frac{(8190 + 10641)}{20.0} = 942^k$$

Case 2 (probable plastic moments @ top and bottom of column)

$$V_{uy} = \frac{(8233 + 10641)}{20.0} = 944^k$$

Case 3 (nominal moment @ 5'-6" from top of column, probable plastic moment @ bottom of column)

$$V_{uy} = \frac{(7704 + 10641)}{14.5} = 1265^k$$

From the above results it can be concluded that if plastic hinges form they probably will form at the top and bottom of the column about the X-axis and at the bottom of the column about the Y-axis.

Since the column section at the bottom is circular and spiral shear reinforcement will be used, the shear in the Y-axis direction will control the design.

$$\therefore V_{u\ max} = 944^k$$

$$H_x^{\ DL+EQ} = 828^k$$

$$H_y^{\ DL+EQ} = 865^k$$

Although Article 1.5.35(G)(1) permits using the lesser value of 865^k for the design shear force V_u, it is desireable to use the design shear forces associated with the development of the column moment strengths. This is particularly so for short structures for which the response of the abutment - abutment foundation material system and its influence on the response of the structure as a whole during an earthquake is so uncertain at this time.

Transverse reinforcement - determine reinforcement for confinement and shear, Articles 1.5.11(B) and 1.5.35(G).

Confinement, using spiral reinforcement, the following three volumetric equations should be satisfied:

$$\rho_s = 0.45 \left(\frac{Ag}{Ac} - 1\right) \frac{f'_c}{f_y} \qquad \text{Eq. (1)}$$

$$\rho_s = 0.45 \left(\frac{Ag}{Ac} - 1\right) \frac{f'_c}{f_y} \left(0.5 + 1.25 \frac{Pe}{f'_c A_g}\right) \qquad \text{Eq. (2)}$$

$$\rho_s = 0.12 \frac{f'_c}{f_y} \left(0.5 + 1.25 \frac{Pe}{f'_c A_g}\right) \qquad \text{Eq. (3)}$$

Equations (2) and (3) apply only to regions of potential plastic hinging.

The confinement requirements for the basic section at the bottom of the column will satisfy the requirements for the remainder of the column.

$$A_g = \frac{\pi (66.0)^2}{4} = 3421.2 \text{ sq.in.}$$

$$A_c = \frac{\pi (62.0)^2}{4} = 3019.1 \text{ sq.in.}$$

$f'_c = 3250$ psi , $f_y = 60000$ psi

Pe = 1209k

Eq. (1) $\rho_s = 0.45 \left(\frac{3421.2}{3019.1} - 1\right) \frac{3250}{60000} = 0.003246$

Eq. (2) $\rho_s = 0.003246 \left(0.5 + 1.25 \frac{1209 \times 1000}{3250 \times 3421.2}\right)$

$= 0.003246 \times 0.63592 = 0.00206$

Eq. (3) $\rho_s = 0.12 \frac{3250}{60000} \times 0.63592 = 0.00413$ <controls

Try #4 spirals @ 3" pitch

$\text{Vol.}_{\text{spiral}} = 0.20 \times 2\pi(33.0 - 2.0 - 0.28) = 38.60$ cu.in.

$\text{Vol.}_{\text{concrete}} = 3.0 \times 3019.1 = 9057.3$ cu.in.

$\rho_s = 38.60/9057.3 = 0.00426 > 0.00413$ ok

Shear, use core section of basic section

$$v_u = \frac{V_u}{b_w d} = \frac{944}{62.0(0.8 \times 62.0)} = 0.307 \text{ ksi}$$

Determine average compressive stress on the core concrete area due to Pe.

$$f_{c\ avg.} = \frac{1209}{3019.1} = 0.400 \text{ ksi}$$

$$0.1f'_c = 0.325 \text{ ksi} < 0.400 \text{ ksi}$$

$$\therefore \text{ can use } v_c = \frac{2(f'_c)^{0.5}}{1000} = 0.114 \text{ ksi}$$

$$v_u - v_c = 0.307 - 0.114 = 0.193 \text{ ksi}$$

$$8\ \frac{(3250)^{0.5}}{1000} = 0.456 \text{ ksi} > 0.193 \text{ ksi} \therefore \text{ Section size is adequate.}$$

$$V_c = v_c b_w d = 0.114 \times 62.0\ (0.8 \times 62.0) = 350^k$$

$$V_{s\ req'd.} = \frac{V_u}{\phi} - V_c = \frac{944}{0.85} - 350 = 761^k$$

Try #5 spiral at 3" pitch

$$V_s = \frac{A_v f_y d}{s} = \frac{2 \times 0.31 \times 60.0 \times 0.8(66.0 - 4.0 - 0.70)}{3.0}$$

$$= 608^k < 761^k \quad \text{N.G.}$$

$$S_{req'd.} : \frac{608}{761} \times 3.0 = 2.40"$$

Try #6 spiral at 3.5" pitch

$$V_s = \frac{2 \times 0.44 \times 60.0 \times 0.8(66.0 - 4.0 - 0.88)}{3.5} = 738^k < 761^k$$

3% under. Say ok.

Check for A_v min. per Article 1.5.10(A)(2)

$$A_{v\ min} = \frac{50 b_w s}{f_y} = \frac{50 \times 62.0 \times 3.5}{50000} = 0.217 \text{ sq.in.}$$

$<< 0.88$ sq.in. ok

- Use #6 spiral @ 3.5" pitch for full length of column and extend into bent cap and footing per Articles 1.5.35(I) and 1.4.6(J) respectively. Spiral may be discontinuous at the bottom bent cap reinforcement and top footing reinforcement, but should be anchored on each side of these levels of horizontal reinforcement.

Piles: Use standard 70 ton piles

Ultimate axial bearing capacity	= 280^k
Ultimate axial uplift capacity	= 112^k
Ultimate lateral resistance	= 30^k except for Group VII = 40^k

Footing:

$f'_c = 3250$ psi
$f_y = 60000$ psi

Determine the pile layout, footing size, and footing reinforcement required to resist the bottom of column forces and moments. Use the centerline of bent (X-axis) and the centerline of column (Y-axis) as the principal axes of the footing.

Minimum footing thickness

26.00"	development length of outer ring of column reinforcement
6.00	additional embedment of inner ring of column reinforcement
3.26	#11 footing reinforcement
6.00	clearance to bottom footing reinforcement
41.26"	

Determine a pile layout that is adequate for Group VII loads and check for other group loads.

Comparison of the dead load plus elastic ARS earthquake forces with the forces resulting from seismic plastic hinging indicates that the latter will be the lesser of the two (Article 1.2.20(f)).

Bottom of column forces resulting from plastic hinging

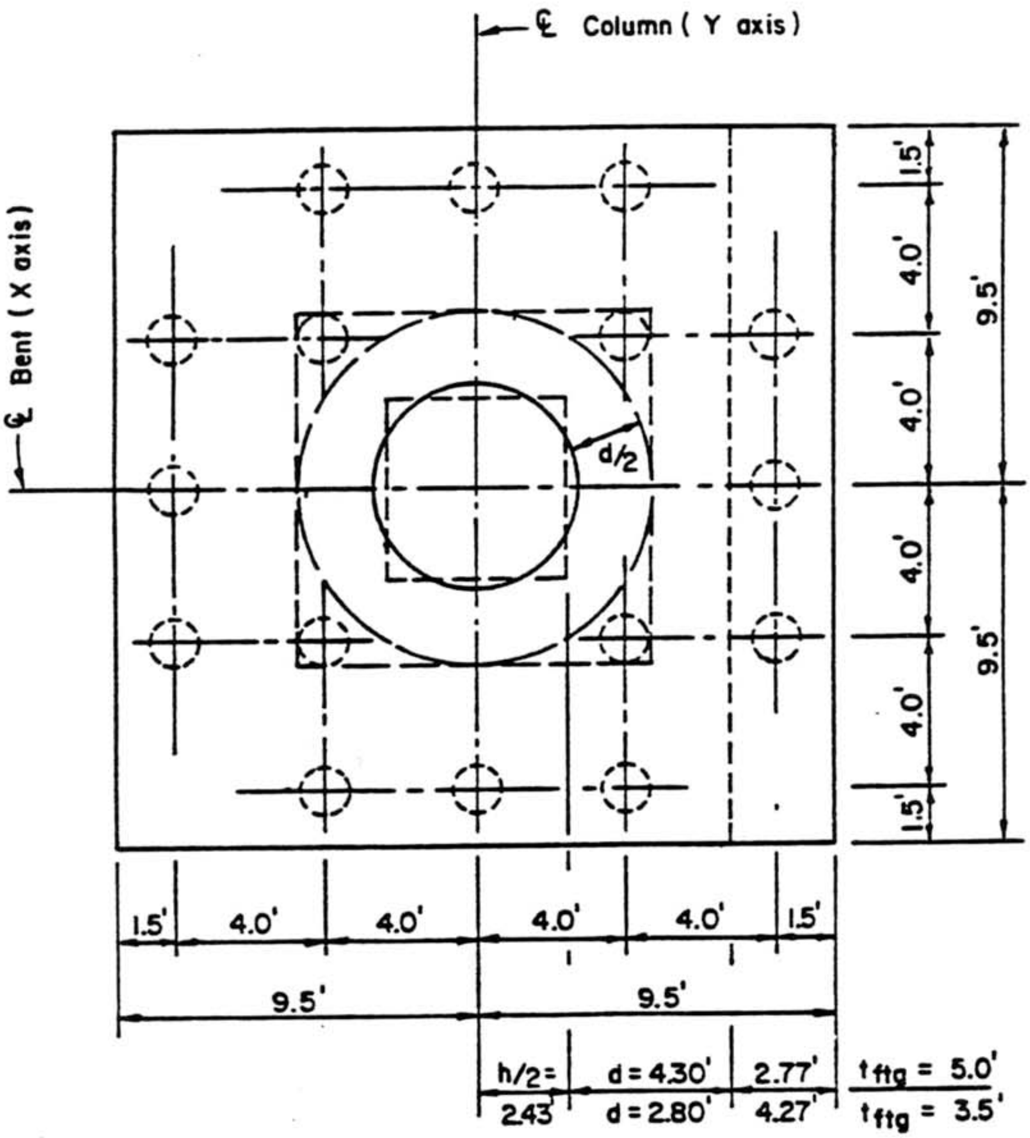
Case 1 (hinging either about X-axis or Y-axis)

$$Pe = 1209^k \quad , \quad M_p = 10641 \text{ ft-kips}$$

Case 2 (Hinging about axis at 45° from x-axis)

$$Pe = 1209^k \quad ; \quad M_{px} = M_{py} = 7524 \text{ ft-kips}$$

Try a 16 pile footing 19.0' x 19.0' x 5.0'



Footing Plan
1/4" = 1'0"

$$P^{DL}_{ftg.} = 5.0 \times 19.0 \times 19.0 \times 0.150 = 270^k$$

$$P^{DL}_{COVER} = (19.0 \times 19.0 - 23.8)\ 2 \times 0.120 = \underline{81^k}$$

$$351k$$

A_{piles} = 16 piles

$$I_{piles} = 6 \times \overline{8.0}^2 = 384$$

(each direction) $8 \times \overline{4.0}^2 = \underline{128}$

512 pile-ft.^{2}

Pile reactions - Group VII

Case 1

$$P_{pile} = \frac{(1209 + 351)}{16} \pm \frac{10641 \times 8.0}{512}$$

$$= 97.5 \pm 166.3 = 263.8^k \text{ max} < 280^k \text{ ok}$$

$$|-68.8^k \text{ min}| < |-112^k| \text{ ok}$$

Case 2

$$P_{pile} = \frac{(1209 + 351)}{16} \pm \frac{7524 \times 8.0}{512} \pm \frac{7524 \times 4.0}{512}$$

$$= 97.5 \pm 117.6 \pm 58.8 = 273.9^k \text{ max} < 280^k \text{ ok}$$

$$|-78.9^k \text{ min}| < |-112^k| \text{ ok}$$

Factored Group I loads (at bottom of column)

Case 1

$$P = 1.3\ (1154 + \frac{1.67}{1.22} \times 224) = 1899^k$$

$$M_x = 1.3\ (216 + \frac{1.67}{1.22} \times 60) = 388 \text{ ft-kips}$$

$$M_y = 1.3\ (0 + \frac{1.67}{1.22} \times 1121) = 1995 \text{ ft-kips}$$

Case 2

$$P = 1.3\ (1154 + \frac{1.67}{1.22} \times 131) = 1733^k$$

$$M_x = 1.3\ (216 + \frac{1.67}{1.22} \times 516) = 1199 \text{ ft-kips}$$

$$M_y = 1.3\ (0 + \frac{1.67}{1.22} \times 654) = 1164 \text{ ft-kips}$$

Case 3

$$P = 1.3\ (1154 + \frac{456}{1.22}) = 1986^k$$

$$M_x = 1.3\ (216 + \frac{136}{1.22}) = 426 \text{ ft-kips}$$

$$M_y = 1.3\ (0 + \frac{3672}{1.22}) = 3913 \text{ ft-kips}$$

Case 4

$$P = 1.3\ (1154 + \frac{269}{1.22}) = 1787^k$$

$$M_x = 1.3\ (216 + \frac{952}{1.22}) = 1295 \text{ ft-kips}$$

$$M_y = 1.3\ (0 + \frac{2179}{1.22}) = 2322 \text{ ft-kips}$$

Case 5

$$P = 1.3\ (1154 + \frac{204}{1.22}) = 1718^k$$

$$M_x = 1.3\ (216 + \frac{694}{1.22}) = 1020 \text{ ft-kips}$$

$$M_y = 1.3\ (0 + \frac{2244}{1.22}) = 2391 \text{ ft-kips}$$

Pile Reactions - Group I loads

Case 1

$$P_{pile} = \frac{(1.3 \times 351 + 1899)}{16} \pm \frac{388 \times 4.0}{512} \pm \frac{1995 \times 8.0}{512}$$

$$= 147.2 \pm 3.0 \pm 31.2 = 181.4^k < 210^k \text{ ok}$$

$$\phi\, P_{n\ pile} = 0.75 \times 280 = 210^k \quad \text{(Article 1.4.6(D))}$$

Case 2

$$P_{pile} = \frac{(1.3 \times 351 + 1733)}{16} \pm \frac{1199 \times 8.0}{512} \pm \frac{1164 \times 4.0}{512}$$

$$= 136.8 \pm 18.7 \pm 9.1 = 164.6^k < 210^k \quad \text{ok}$$

Case 3

$$P_{pile} = \frac{(1.3 \times 351 + 1986)}{16} \pm \frac{426 \times 4.0}{512} \pm \frac{3913 \times 8.0}{512}$$

$$= 152.6 \pm 3.3 \pm 61.1 = 217.0^k > 210^k$$

3.3% over, say ok

Case 4

$$P_{pile} = \frac{(1.3 \times 351 + 1787)}{16} \pm \frac{1295 \times 4.0}{512} \pm \frac{2322 \times 8.0}{512}$$

$$= 140.2 + 10.1 + 36.3 = 186.6^k < 210^k \quad \text{ok}$$

Case 5

$$P_{pile} = \frac{(1.3 \times 351 + 1718)}{16} \pm \frac{1020 \times 4.0}{512} \pm \frac{2391 \times 8.0}{512}$$

$$= 135.9 \pm 8.0 \pm 37.4 = 181.3^k < 210^k \quad \text{ok}$$

Determine footing shear requirements.

Equivalent square column section (Article 1.4.6(F))

$$h^2 = \frac{\pi(66.0)^2}{4} = 3421.2; \; h = 58.5" \; (4.87')$$

From a comparison of pile reactions it can be determined that Group VII case 1 loads will control.

Assume #11 bars in bottom mat of reinforcement

$$d_{min}^{\#11} = 60.0 - 6.0 - 1.5 \times 1.63 = 51.56" \; (4.30')$$

Shear at section through footing at distance d from face of column (Article 1.5.35(F)(1)(a))

$$\begin{aligned} V_u &= 3 \times 263.8 &&= 791.4 \\ &- 5.0 \times 2.77 \times 19.0 \times 0.150 &&= -39.5 \\ &- 2.0 \times 2.77 \times 19.0 \times 0.120 &&= -12.6 \\ & &&\overline{739.3^k} \end{aligned}$$

V_u allowable on section $= \phi\ 10(f'_c)^{0.5}\ b_w d$; (Articles 1.5.35(B)(1) and 1.5.35(C)(6))

$$= 0.85 \times 10 \frac{(3250)^{0.5}}{1000} (19.0 \times 12)\ 51.56 = 5696^k$$

$$V_c = 2 \frac{(3250)^{0.5}}{1000} (19.0 \times 12)\ 51.56 = 1340^k$$

$\phi V_c = 0.85 \times 1340 = 1139^k > 739.3^k$ $\therefore$ do not need shear reinforcement

Shear at section concentric with and at a distance d/2 from the face of column (Article 1.3.35(F)(1)(b)).

Use section d/2 from face of actual circular column section.

Neglect tensile pile reactions.

$V_u =$	3×263.8	=	791.4
	4×180.6	=	722.4
	2×97.5	=	195.0
	4×14.4	=	57.6
	$-(19.0^2 - 0.25\,\pi\,(5.5 + 4.3)^2)\ 5.0 \times 0.150$	=	$-$ 214.2
	$-($ " " " $)\ 2.0 \times 0.120$	=	$-$ 68.5
			1483.7^k

V_u allowable on section $= \phi\ 6(f'_c)^{0.5}\ b_o d$; (Article 1.5.35(F)(4))

$$= 0.85 \times 6 \frac{(3250)^{0.5}}{1000} \pi \times 9.8 \times 4.30 \times 144 = 5543^k > 1484^k$$

V_c without shear reinforcement $= 4(f'_c)^{0.5}\ b_o d$; (Article 1.5.35(F)(3))

$$= 4 \frac{(3250)^{0.5}}{1000} \pi \times 9.8 \times 4.30 \times 144 = 4347^k$$

$\phi V_c = 0.85 \times 4347 = 3695^k > 1484^k$ ok

do not need shear reinforcement

Determine minimum footing thickness that is adequate for shear without shear reinforcement.

Try 3.5' footing thickness

$$d_{min}^{\#11} = 42.0 - 6.0 - 1.5 \times 1.63 = 33.56" \ (2.80')$$

Pile reactions are based on the dead load of a 5.0' thick footing, therefore use this dead load for reducing the applied moments and shears.

Shear of section through footing at distance d from face of column.

$$
\begin{aligned}
V_u &= 3 \times 263.8 &&= 791.4 \\
&\quad - 5.0 \times 4.27 \times 19.0 \times 0.150 &&= -\ 60.8 \\
&\quad - 2.0 \times 4.27 \times 19.0 \times 0.120 &&= -\ \underline{19.5} \\
& && \quad\ 711.1^k
\end{aligned}
$$

$$\phi\ V_c = 0.85 \times 2\ \frac{(3250)^{0.5}}{1000}\ (19.0 \times 12)\ 33.56 = 742^k > 711^k \text{ ok}$$

Shear at section concentric with and at a distance d/2 from the face of column.

$$
\begin{aligned}
V_u &= &&= 1766.4 \\
&\quad - (19.0^2 - 0.25 \quad (5.5 + 2.8)^2)\ 5.0 \times 0.150 &&= -\ 230.2 \\
&\quad - (\ \text{"} \qquad\qquad \text{"} \qquad\qquad \text{"}\ )\ 2.0 \times 0.120 &&= -\ \underline{73.6} \\
& && \quad 1462.6^k
\end{aligned}
$$

$$\phi\ V_c = 0.85 \times 4\ \frac{(3250)^{0.5}}{1000} \times 8.3 \times 2.80 \times 144 = 2038^k > 1463^k \text{ ok}$$

$t_{ftg.\ min.}$ 3'-6" ; use minimum shear reinforcement

- Determine flexural reinforcement required (Articles 1.4.6(G), 1.5.7, 1.5.32 and 1.5.37)

Bottom of footing flexural reinforcement -

Section at face of column

$$
\begin{aligned}
M_u &= 3 \times 263.8\ (8.0 - 2.43) &&= 4408 \\
&\quad 4 \times 180.6\ (4.0 - 2.43) &&= 1134 \\
&\quad - 5.0 \times 7.07 \times 19.0 \times 0.150 \times \frac{7.07}{2} &&= -\ 356 \\
&\quad - 2.0 \times 7.07 \times 19.0 \times 0.120 \times \frac{7.07}{2} &&= \underline{-\ 114} \\
& &&\ 5072 \text{ ft-kips}
\end{aligned}
$$

$$1.2\ M_{cr} = 1.2 \times 7.5\ (f'_c)^{0.5}\ \frac{I_g}{y_t}$$

$$= 1.2 \times 7.5\ \frac{(3250)^{0.5}}{1000}\ \frac{\frac{1}{12}(19.0 \times 12)(42.0)^3}{0.5 \times 42.0} \div 12 = 2866 \text{ ft-kips}$$

$M_u > 1.2\ M_{cr}\ \therefore\ A_{s\ req'd.} > A_{s\ min.}$

try A_s = #9 @ 6 $\pm$ = 39 - #9 = 39.0 sq.in.

$$d_{min}^{\#9} = 42.0 - 6.0 - 1.5 \times 1.25 = 34.12"$$

$$a = \frac{A_s f_y}{0.85\ f'_c b} = \frac{39.0 \times 60.0}{0.85 \times 3.25 \times 19.0 \times 12} = 3.72"$$

$$\phi M_n = \phi A_s f_y\ (d - \frac{a}{2}) = 1.0 \times 39.0 \times 60.0(34.12 - \frac{3.72}{2}) \div 12$$

$$= 6291 \text{ ft-kips} > 5072 \text{ ft-kips} \quad \text{ok}$$

$$\rho_s = 39.0/(19.0 \times 12 \times 34.12) = 0.005$$

$$0.75\ \rho_b = 0.75\ \frac{0.85 \times 0.85 \times 3250}{60000}\ (\frac{87000}{87000 + 60000}) = 0.0174$$

$$0.005 < 0.0174 \quad \text{ok}$$

Check serviceability requirements of Article 1.5.39

Determine pile reactions under service loads Group I

Case 1

$$P = 1154 + 271 + \frac{224}{1.22} = 1609^k$$

$$M_x = 216 + \frac{60}{1.22} = 265 \text{ ft-kips}$$

$$M_y = 0 + \frac{1121}{1.22} = 919 \text{ ft-kips}$$

$$P_{pile} = \frac{1609}{16} \pm \frac{265 \times 4.0}{512} \pm \frac{919 \times 8.0}{512}$$

$$= 100.6 \pm 2.1 \pm 14.4 = 117.1^k \text{ max} < \underline{\text{controls}}$$

Case 2

$$P = 1154 + 271 + \frac{131}{1.22} = 1532^k$$

$$M_x = 216 + \frac{516}{1.22} = 639 \text{ ft-kips}$$

$$M_y = 0 + \frac{654}{1.22} = 536 \text{ ft-kips}$$

$$P_{pile} = \frac{1532}{16} \pm \frac{639 \times 8.0}{512} \pm \frac{536 \times 4.0}{512}$$

$$= 95.8 \pm 10.0 \pm 4.2 = 110.0^k \text{ max.}$$

$$\begin{aligned}
M = \; & 3 \times 115.0\ (8.0 - 2.43) && = 1922 \\
& 4 \times 107.8\ (4.0 - 2.43) && = 677 \\
& - 3.5 \times 7.07 \times 19.0 \times 0.150 \times 3.53 && = -249 \\
& - 2.0 \times 7.07 \times 19.0 \times 0.120 \times 3.53 && = \underline{-114} \\
& && \ 2236 \text{ ft-kips}
\end{aligned}$$

$b = 228.0"$

$d = 34.12"$

$n = 9$

$f_c = 0.86$ ksi

$f_s = 22.1$ ksi < 24.0 ksi ok

Use #9 @ 6 $\pm$ total 39 each direction

Top of footing flexural reinforcement (Article 1.4.6(G))

Use #9 @ 12 $\pm$ total 20 each direction

Minimum shear reinforcement

Use #5 J @ 12 placed per Article 1.4.6(H).

Compare the available lateral resistance of the soil and foundation system with the horizontal seismic forces at the bottom of the column. It is not a requirement to provide a bent foundation design to resist the horizontal seismic forces, but if the available resistance is significantly less than the seismic forces then large permanent displacements of the foundation may result after a large earthquake.

Soil parameters of soil in which footing is embedded

$\phi = 28°$

$\gamma = 120$ pcf

$K_a = 0.34$

$K_p = 5.5 \times 0.872 = 4.79$

$P_{p_{net}} \sim 0.5(4.79 - 0.34)\ 0.120\ (2.0 + 3.5)^2 \times 19.0 = 153^k$

$H_{n\ piles} \sim 16 \times 40 = 640^k$

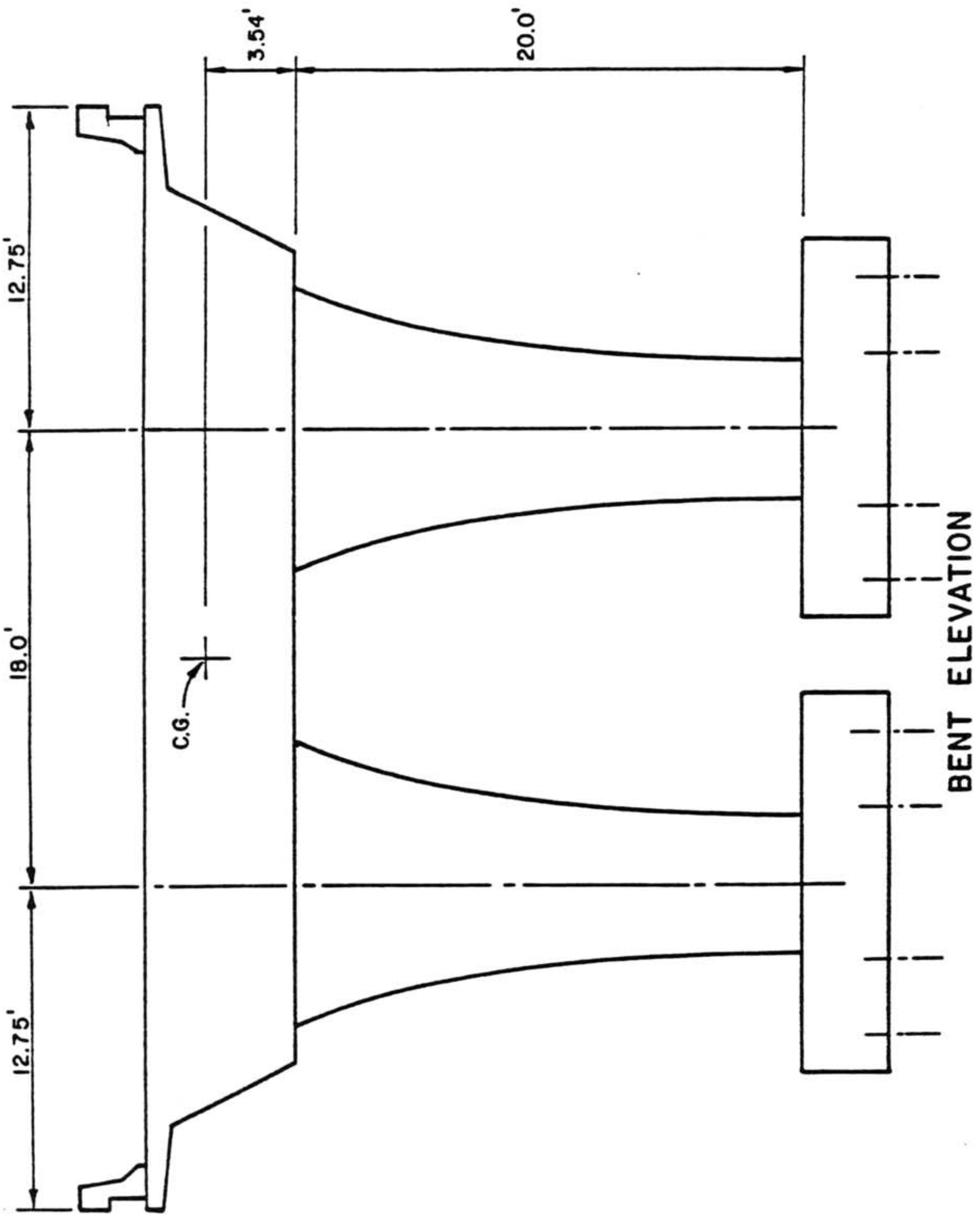
$H_{n\ foundation} \sim 153 + 640 = 793^k < 944^k$

$\sim$ 16% under seismic horizontal force

Increasing the embedment of the footing would increase the resistance available and reduce the seismic force (column shear) so that it would be possible to reduce the likelihood of large permanent bent footing displacements after a large earthquake.

Example Problem 2 - Two Column Bent W/Pile Footings

Problem: Design the columns and pile footings for the structure shown below. This is the same problem which is used as Problem Set No. 1 of Reinforced Concrete Section of the Office of Structures Design correspondence course.



Column loads: For illustrative purposes, only dead load, live load and seismic loads will be considered. The X-axis equals the centerline of bent and the Y-axis equals the centerline of column.

Dead load (service level)

$P^{DL}_{TOP\ COL} = 815^k$ $\quad$ $P^{DL}_{BOT\ COL} = 914^k$

$M_x{}^{DL}_{TOP\ COL} = -1845$ ft-kips $\quad$ $M_x{}^{DL}_{BOT\ COL} = 159$ ft-kips

$M_y{}^{DL}_{TOP\ COL} = 1150$ ft-kips $\quad$ $M_y{}^{DL}_{BOT\ COL} = -360$ ft-kips

Live load + impact (service level); Impact = 21%

Case 1

$P^{LL+I} = 256^k$

$M_x{}^{LL+I}_{TOP\ COL} = -391$ ft-kips $\quad$ $M_x{}^{LL+I}_{BOT\ COL} = 159$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 162$ ft-kips $\quad$ $M_x{}^{LL+I}_{BOT\ COL} = -3$ ft-kips

Case 2

$P^{LL+I} = 131^k$

$M_x{}^{LL+I}_{TOP\ COL} = -1864$ ft-kips $\quad$ $M_x{}^{LL+I}_{BOT\ COL} = 756$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 83$ ft-kips $\quad$ $M_y{}^{LL+I}_{BOT\ COL} = -2$ ft-kips

Case 3

$P^{LL+I} = 130^k$

$M_x{}^{LL+I}_{TOP\ COL} = -234$ ft-kips $\quad$ $M_x{}^{LL+I}_{BOT\ COL} = 95$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 601$ ft-kips $\quad$ $M_y{}^{LL+I}_{BOT\ COL} = -188$ ft-kips

Live load + impact (for factored level); Impact = 21%

Case 1 (1.15 x 1 lane P + 1 lane H)

$P^{LL+I} = 576^k$

$M_x{}^{LL+I}_{TOP\ COL} = -982$ ft-kips $M_x{}^{LL+I}_{BOT\ COL} = 399$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 812$ ft-kips $M_y{}^{LL+I}_{BOT\ COL} = -102$ ft-kips

Case 2 (1.15 x 1 lane P + 1 lane H)

$P^{LL+I} = 383^k$

$M_x{}^{LL+I}_{TOP\ COL} = -4304$ ft-kips $M_x{}^{LL+I}_{BOT\ COL} = 1746$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 595$ ft-kips $M_y{}^{LL+I}_{BOT\ COL} = -80$ ft-kips

Case 3 (1.15 x 1 lane P + 1 lane H)

$P^{LL+I} = 450^k$

$M_x{}^{LL+I}_{TOP\ COL} = -825$ ft-kips $M_x{}^{LL+I}_{BOT\ COL} = 335$ ft-kips

$M_y{}^{LL+I}_{TOP\ COL} = 1251$ ft-kips $M_y{}^{LL+I}_{BOT\ COL} = -287$ ft-kips

Seismic load (ARS forces and moments before and after application of z = 8 factor)

Transverse earthquake motion (RMS results)

$H_x{}^{EQ}_{TOP\ COL} = 880^k$ $H_y{}^{EQ}_{TOP\ COL} = 32^k$ (Horizontal forces)

$H_x{}^{EQ}_{BOT\ COL} = 921^k$ $H_y{}^{EQ}_{BOT\ COL} = 34^k$

Case 1

$P^{EQ}_{TOP\ COL} = \pm 930^k$

$M_x{}^{EQ}_{TOP\ COL} = 328 / 8 = 41$ ft-kips

$M_y{}^{EQ}_{TOP\ COL} = 8240 / 8 = 1030$ ft-kips

Case 2

$P_{\text{BOT COL}}^{\text{EQ}} = \pm 932^k$

$M_{x\,\text{BOT COL}}^{\text{EQ}} = 334 / 8 = 42$ ft-kips

$M_{y\,\text{BOT COL}}^{\text{EQ}} = 9832 / 8 = 1229$ ft-kips

Longitudinal earthquake motion (RMS results)

$H_{x\,\text{TOP COL}}^{\text{EQ}} = 28^k$ $H_{y\,\text{TOP COL}}^{\text{EQ}} = 461^k$ (Horizontal forces)

$H_{x\,\text{BOT COL}}^{\text{EQ}} = 29^k$ $H_{y\,\text{BOT COL}}^{\text{EQ}} = 492^k$

Case 3

$P_{\text{TOP COL}}^{\text{EQ}} = \pm 202^k$

$M_{x\,\text{TOP COL}}^{\text{EQ}} = 4597 / 8 = 575$ ft-kips

$M_{y\,\text{TOP COL}}^{\text{EQ}} = 384 / 8 = 48$ ft-kips

Case 4

$P_{\text{BOT COL}}^{\text{EQ}} = \pm 202^k$

$M_{x\,\text{BOT COL}}^{\text{EQ}} = 5050 / 8 = 631$ ft-kips

$M_{y\,\text{BOT COL}}^{\text{EQ}} = 191 / 8 = 24$ ft-kips

Combined earthquake motion

Case 5 ($1.0\ EQ_L + 0.3\ EQ_T$)

$P_{\text{TOP COL}}^{\text{EQ}} = 202 + 0.3 \times 930 = 481^k$

$M_{x\,\text{TOP COL}}^{\text{EQ}} = 4597 + 0.3 \times 328 = 4695 / 8 = 587$ ft-kips

$M_{y\,\text{TOP COL}}^{\text{EQ}} = 384 + 0.3 \times 8240 = 2856 / 8 = 357$ ft-kips

Case 6 (1.0 EQ_L + 0.3 EQ_T)

$$P^{EQ}_{BOT\ COL} = 202 + 0.3 \times 932 = 482^k$$

$$M_x{}^{EQ}_{BOT\ COL} = 5050 + 0.3 \times 334 = 5150 / 8 = 644 \text{ ft-kips.}$$

$$M_y{}^{EQ}_{BOT\ COL} = 191 + 0.3 \times 9832 = 3141 / 8 = 393 \text{ ft-kips}$$

Case 7 (0.3 EQ_L + 1.0 EQ_T)

$$P^{EQ}_{TOP\ COL} = 0.3 \times 202 + 930 = 991^k$$

$$M_x{}^{EQ}_{TOP\ COL} = 0.3 \times 4597 + 328 = 1707 / 8 = 213 \text{ ft-kips}$$

$$M_y{}^{EQ}_{TOP\ COL} = 0.3 \times 384 + 8240 = 8355 / 8 = 1044 \text{ ft-kips}$$

Case 8 (0.3 EQ_L + 1.0 EQ_T)

$$P^{EQ}_{BOT\ COL} = 0.3 \times 202 + 932 = 993^k$$

$$M_x{}^{EQ}_{BOT\ COL} = 0.3 \times 5050 + 334 = 1849 / 8 = 231 \text{ ft-kips}$$

$$M_y{}^{EQ}_{BOT\ COL} = 0.3 \times 191 + 9832 = 9889 / 8 = 1236 \text{ ft-kips}$$

Columns:

Geometry - Standard architectural column Type 2R

Clear height = 20'-0"
Length of top flare section = 16'-6"
t = 5'-6"

Longitudinal reinforcement - determined by using YIELD program with reinforcement placement controlled by basic section at bottom of column.

$A_{s\ TOP}$ = 54-#9 = 54.00 sq.in.

36-#9 in outer ring for full length column

18-#9 in inner ring for top 2/3 column

$A_{s\ BOTTOM}$ = 36-#9 = 36.0 sq.in.

$A_{g\ TOP\ COL\ ACTUAL}$ = 7777.19 sq.in.

ρ_s = 54.0/7777.19 = 0.0069

$A_{g\ TOP\ COL\ DESIGN}$ = 5401.19 sq.in. (Group loads other than Group VII)

ρ_s = 54.0/5401.19 = 0.0100

$A_{g\ TOP\ COL\ DESIGN}$ = 3421.19 sq.in. (Group VII loads)

ρ_s = 54.0/3421.19 = 0.0158

$A_{g\ BOT\ COL\ ACTUAL\ \&\ DESIGN}$ = 3421.19 sq.in.

ρ_s = 36.0/3421.19 = 0.0105

Probable plastic moment strength -

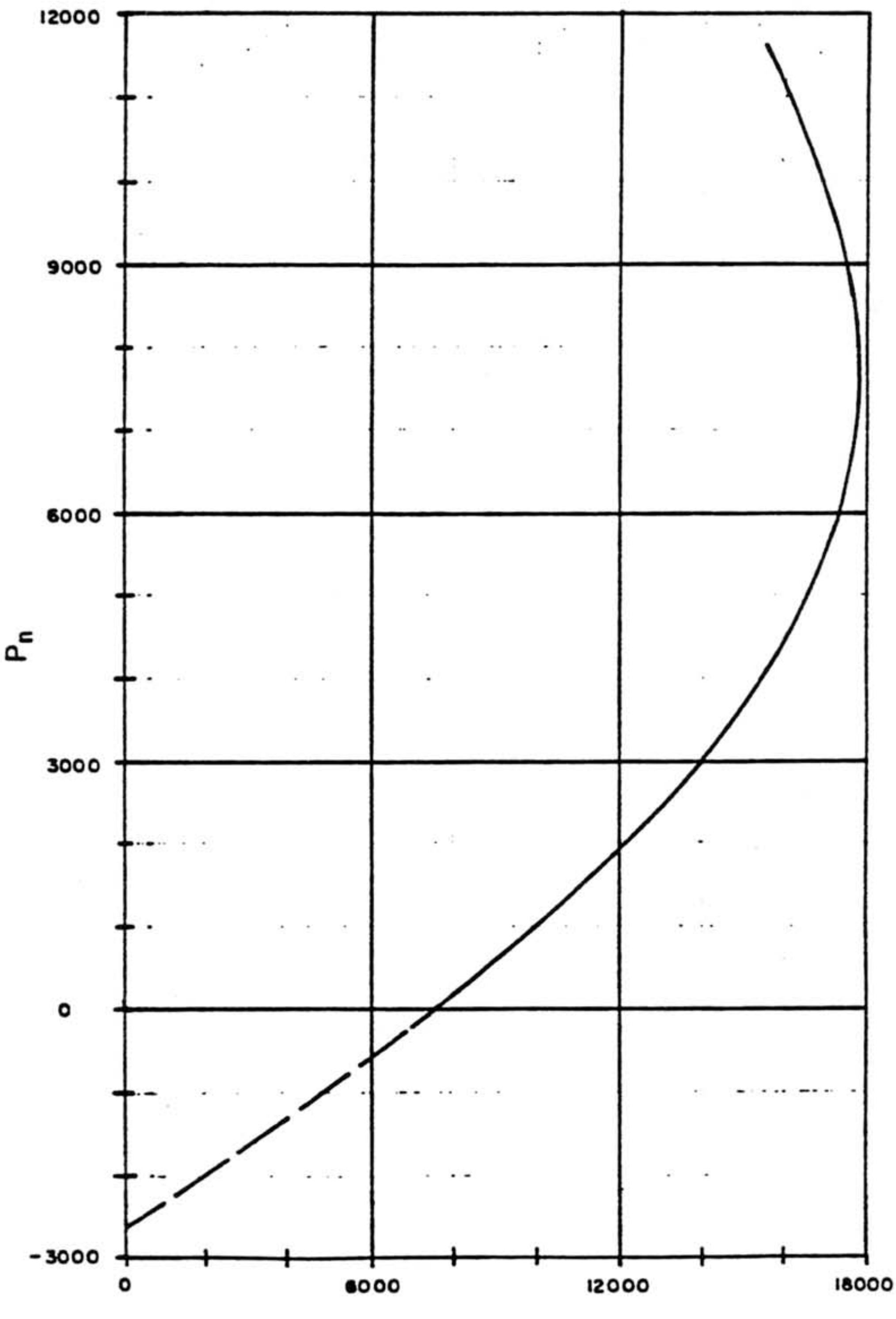
f_c' = 3250 psi , $\epsilon_{c\ max}$ = 0.003 in./in.

f_y = 60000 psi , E_s = 29000000 psi

$M_p = 1.3\ M_n$

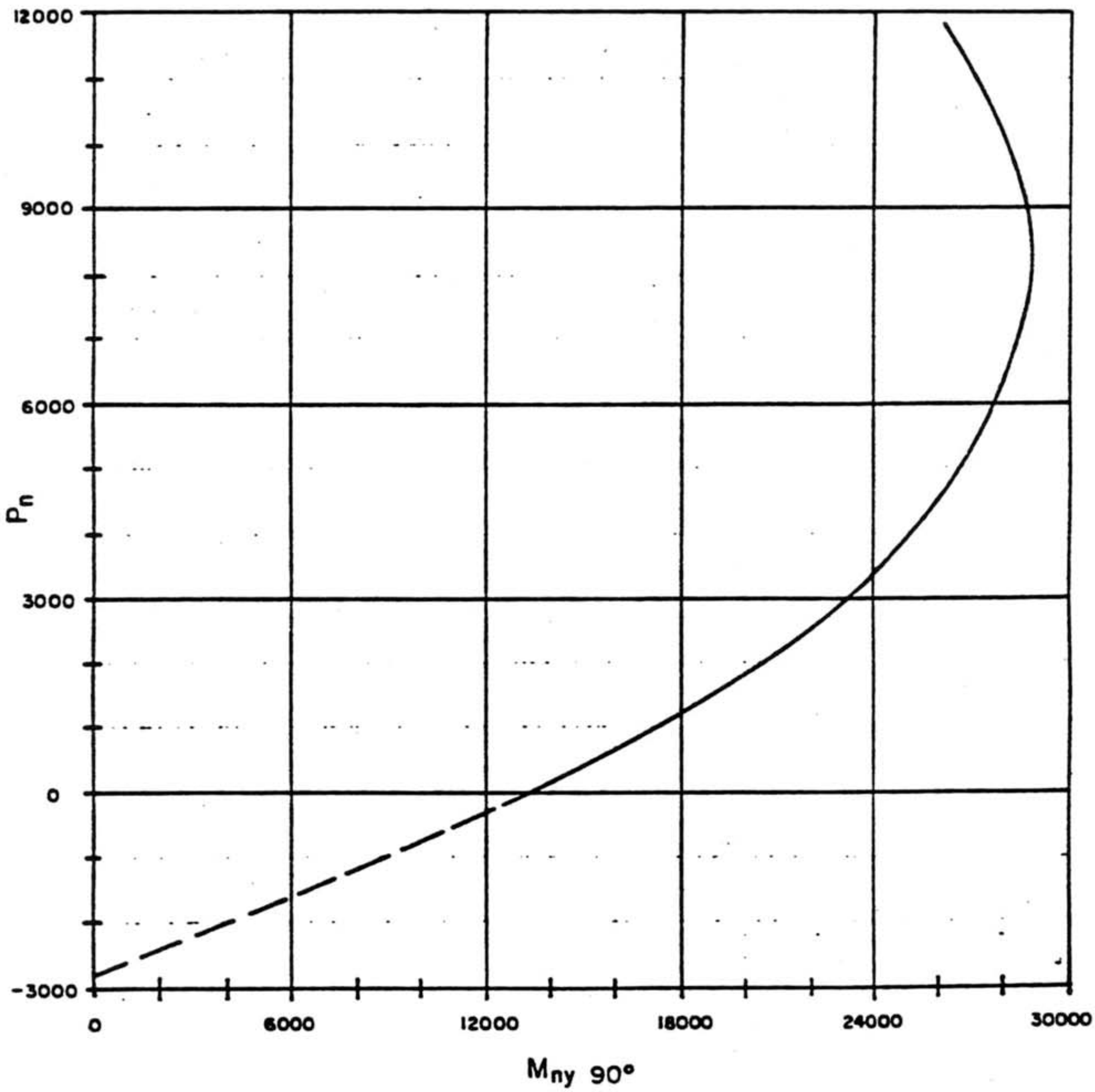
Full Flare Section @ Top of Column

A_s = 54-#9 = 54.0 sq.in.



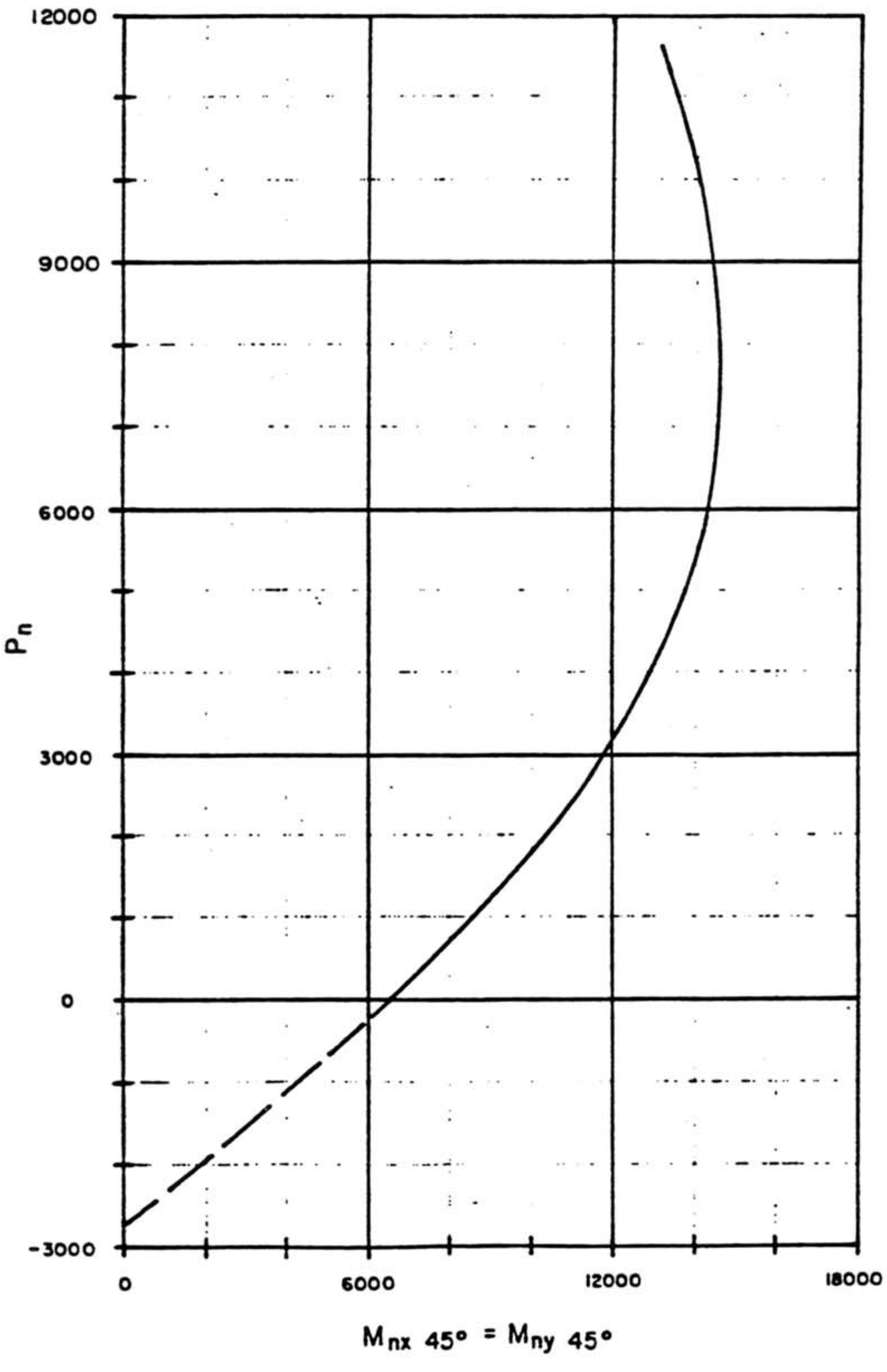
Full Flare Section @ Top of Column

A_s = 54-#9 = 54.0 sq.in.



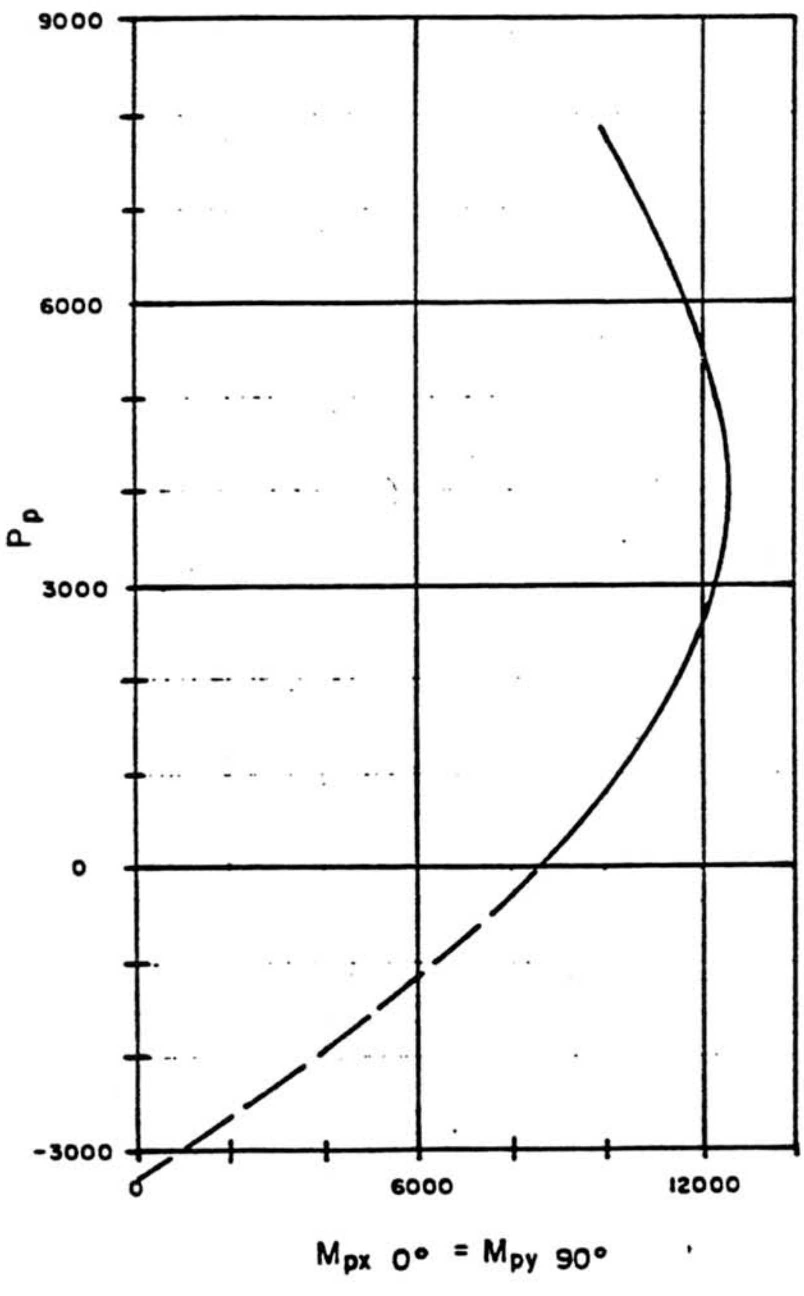
Full Flare Section @ Top of Column

A_s = 54-#9 = 54.0 sq.in.



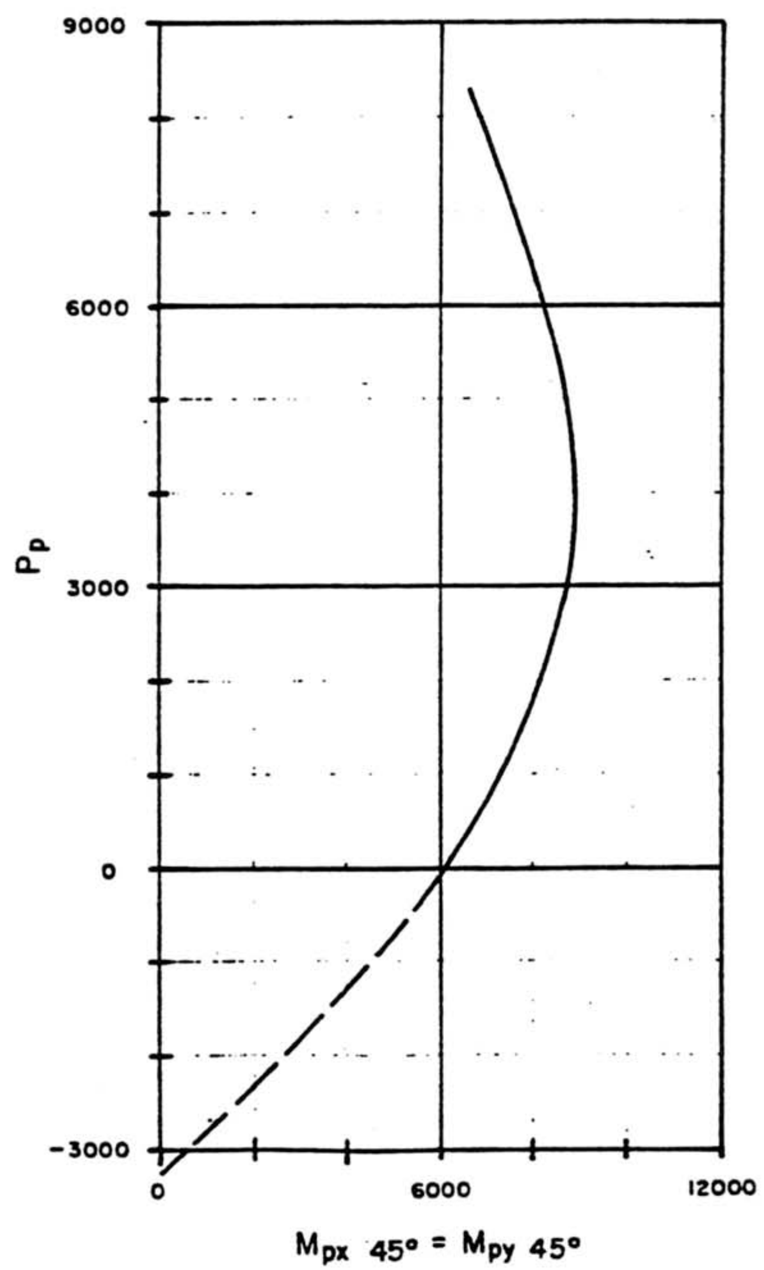
Basic Section @ Top of Column

A_s = 54-#9 = 54.0 sq.in.



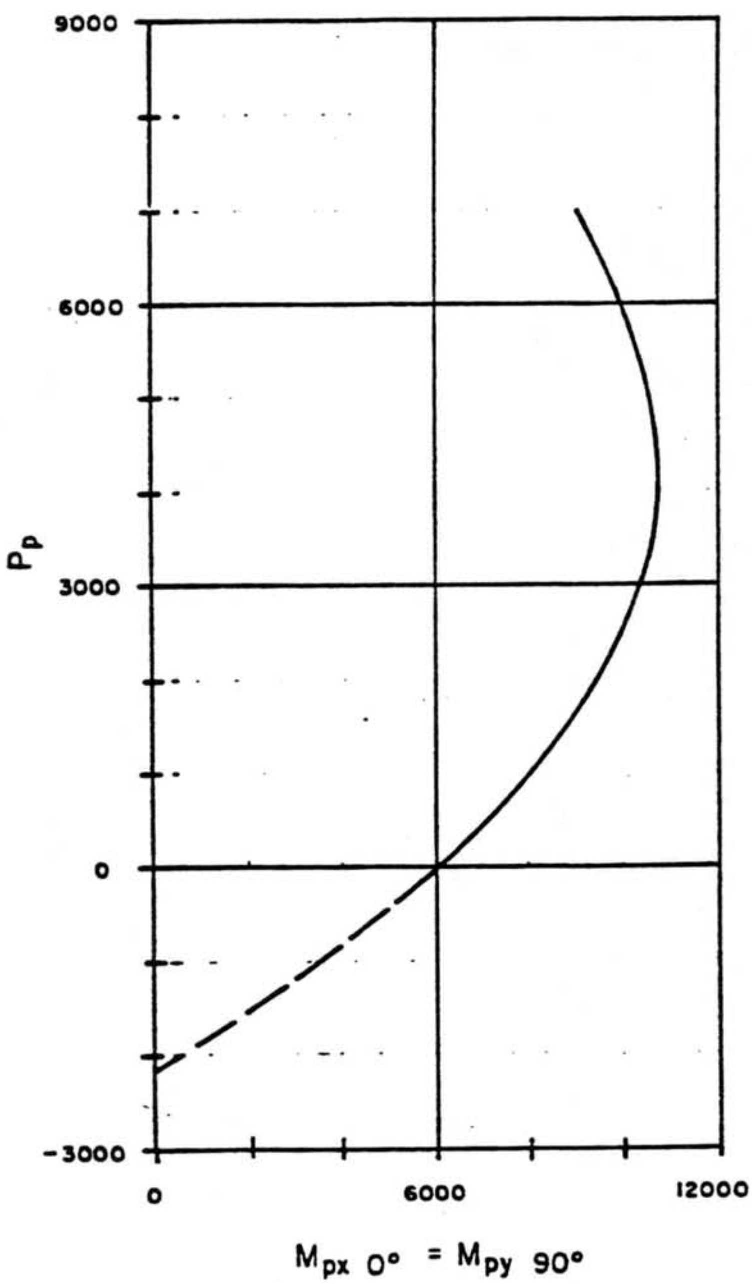
Basic Section @ Top of Column

A_s = 54-#9 = 54.0 sq.in.



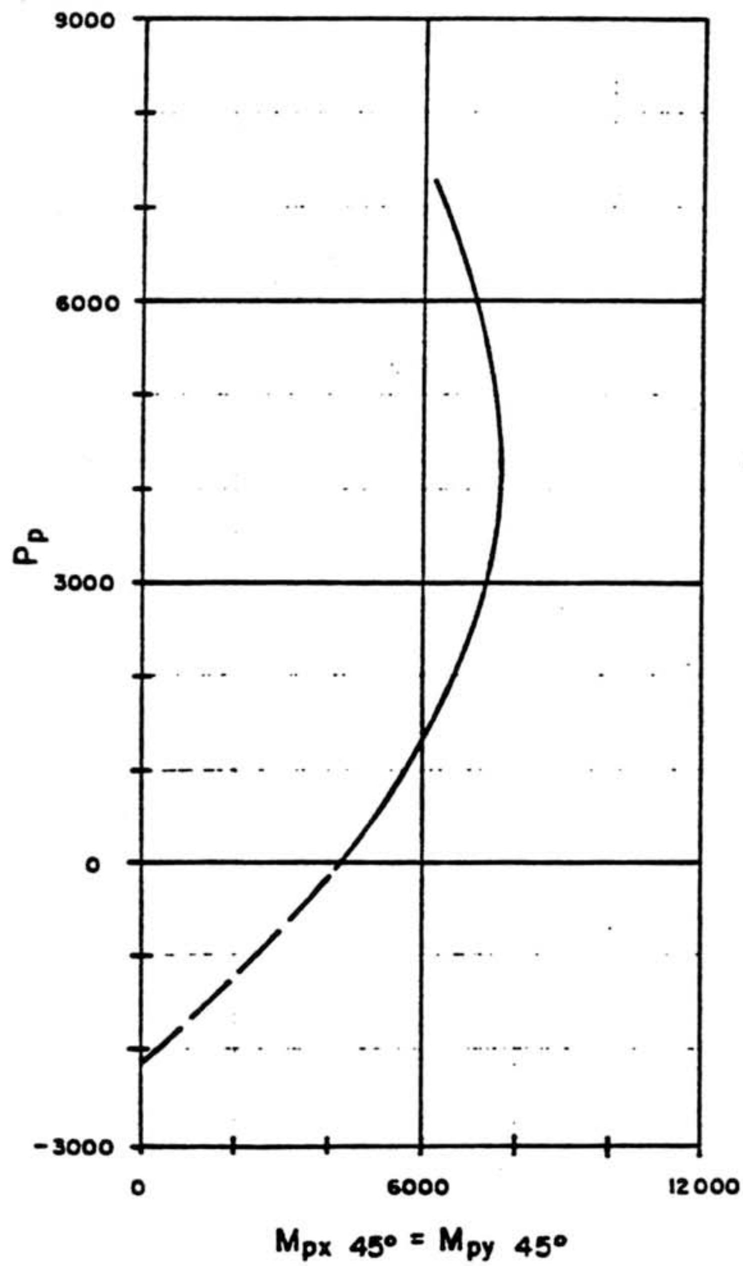
Basic Section @ Bottom of Column

A_s = 36-#9 = 36.0 sq.in.



Basic Section @ Bottom of Column

A_s = 36-#9 = 36.0 sq.in.



Column Shear

Determine the maximum column shears considering that the nominal moment strengths can be developed in the gross flare sections of the columns and that the probable plastic moment strengths can be developed in the basic column section. Consider these moments developing about the X-axis, Y-axis and on axis 45° from the X-axis. Use the dead load axial force plus the change in axial force due to the development of the component of moments about the Y-axis (i.e., take into consideration the effects of transverse overturning).

Moment strengths - Use the interaction curves developed from YIELD program output.

Top of column

$P^{DL} = 815^k$

$M_{nx\ 0°} = 9600$ ft-kips $\qquad M_{px\ 0°} = 10000$ ft-kips

$M_{ny\ 90°} = 16700$ ft-kips $\qquad M_{py\ 90°} = 10000$ ft-kips

Bottom of column

$P^{DL} = 914^k$

$M_{px\ 0°} = M_{py\ 90°} = 7900$ ft-kips

$$V_{ux} = \frac{(M_{ny\ 90°\ TOP} + M_{py\ 90°\ BOT})}{\text{clear column height}} = \frac{(16700 + 7900)}{20.0} = 1230^k$$

$$V_{uy} = \frac{(M_{px\ 0°\ TOP} + M_{px\ 0°\ BOT})}{\text{clear column height}} = \frac{(10000 + 7900)}{20.0} = 895^k$$

$$\Sigma V_{ux\ col} = 2 \times 1230 = 2460^k$$

$$M_y^{OT} = \Sigma V_{ux\ col}\ (\text{clear column height} + y_b \text{ of superstructure})$$

$$= 2460\ (20.0 + 3.54) = 57900 \text{ ft-kips}$$

$$P^{OT} = \pm \frac{M_y^{OT} - \Sigma M_{py\ 90°\ BOT}}{\text{column spacing}} = \pm \frac{57900 - 2 \times 7900}{18.0} = \pm 2339^k$$

Note: The above determination of axial column forces associated with the column moment strength is valid only for 2 column bents with equal length columns. For bents with more than 2 columns and/or for bents with significantly different length columns within the bent, use an appropriate analysis for determining axial column forces associated with the column moment strengths.

Determine new moment strengths, shears and axial forces using P^{DL+OT}.

Top of column

$P_{max}^{DL+OT} = 815 + 2339 = 3154^k$

$M_{ny\ 90°} = 23600$ ft-kips $\qquad M_{py\ 90°} = 12400$ ft-kips

$P_{min}^{DL+OT} = 815 - 2339 = -1524^k$

$M_{ny\ 90°} = 6600$ ft-kips $\qquad M_{py\ 90°} = 5000$ ft-kips

Bottom of column

$P_{max}^{DL+OT} = 914 + 2339 = 3253^k$

$M_{py\ 90°} = 10600$ ft-kips

$P_{min}^{DL+OT} = 914 - 2339 = -1425^k$

$M_{py\ 90°} = 2200$ ft-kips

$$V_{ux\ max} = \frac{(23600 + 10600)}{20.0} = 1710^k$$

$$V_{ux\ min} = \frac{(6600 + 2200)}{20.0} = 440^k$$

$\Sigma V_{ux\ columns} = 1710 + 440 = 2150^k$, ~13% less than previous $\Sigma V_{ux\ columns} = 2460^k$

$M_y^{OT} = 2150 \times 23.54 = 50611$ ft-kips

$$P^{OT} = \pm \frac{50611 - (10600 + 2200)}{18.0} = \pm 2100^k$$

Determine new moment strengths, shears and axial forces using revised P^{DL+OT}.

Top of column

$P_{max}^{DL+OT} = 815 + 2100 = 2915^k$

$M_{ny\ 90°} = 23000$ ft-kips $\qquad M_{py\ 90°} = 12300$ ft-kips

$P_{min}^{DL+OT} = 815 - 2100 = -1285^k$

$M_{ny\ 90°} = 7700$ ft-kips $\qquad M_{py\ 90°} = 5700$ ft-kips

Bottom of column

$P_{max}^{DL+OT} = 914 + 2100 = 3014^k$

$M_{py\ 90°} = 10400$ ft-kips

$P_{min}^{DL+OT} = 914 - 2100 = -1186^k$

$M_{py\ 90°} = 3000$ ft-kips

$$V_{ux\ max} = \frac{(23000 + 10400)}{20.0} = 1670^k$$

$$V_{ux\ min} = \frac{(7700 + 3000)}{20.0} = 535^k$$

$\Sigma V_{ux\ columns} = 1670 + 535 = 2205^k$, ~2.6% greater than previous ΣV_{ux} columns = 2150^k say close enough

Determine moment strengths, shears and axial forces due to yielding of columns at top and bottom due to bending about an axis 45° from the X-axis.

Top of column

$P^{DL} = 815^k$

$M_{nx\ 45°} = M_{ny\ 45°} = 8250$ ft-kips

$M_{px\ 45°} = M_{py\ 45°} = 7100$ ft-kips

Bottom of column

$$P^{DL} = 914^k$$

$$M_{px\ 45°} = M_{py\ 45°} = 5580 \text{ ft-kips}$$

$$V_{ux} = V_{uy} = \frac{(8250 + 5580)}{20.0} = 692^k$$

$$\Sigma V_{ux\ columns} = 2 \times 692 = 1384^k$$

$$M_y^{OT} = 1384\ (20.0 + 3.54) = 32579 \text{ ft-kips}$$

$$P^{OT} = \pm \frac{32579 - 2 \times 5580}{18.0} = \pm 1190^k$$

Determine new moment strengths, shears and axial forces using P^{DL+OT}.

Top of column

$$P_{max}^{DL+OT} = 815 + 1190 = 2005^k$$

$$M_{nx\ 45°} = M_{ny\ 45°} = 10400 \text{ ft-kips}$$

$$M_{px\ 45°} = M_{py\ 45°} = 8200 \text{ ft-kips}$$

$$P_{min}^{DL+OT} = 815 - 1190 = -375^k$$

$$M_{nx\ 45°} = M_{ny\ 45°} = 5800 \text{ ft-kips}$$

$$M_{px\ 45°} = M_{py\ 45°} = 5600 \text{ ft-kips}$$

Bottom of column

$$P_{max}^{DL+OT} = 914 + 1190 = 2104^k$$

$$M_{px\ 45°} = M_{py\ 45°} = 6800 \text{ ft-kips}$$

$$P_{min}^{DL+OT} = 914 - 1190 = -276^k$$

$$M_{px\ 45°} = M_{py\ 45°} = 3800 \text{ ft-kips}$$

$$V_{ux\ max} = V_{uy\ max} = \frac{(10400 + 6800)}{20.0} = 860^k$$

$$V_{ux\ min} = V_{uy\ min} = \frac{(5800 + 3800)}{20.0} = 480^k$$

$\Sigma V_{ux\ columns} = 860 + 480 = 1340^k$, ~ 3.2% less than previous $\Sigma V_{ux\ columns} = 1384^k$
say close enough

$$V_{u\ max} = (860^2 + 860^2)^{0.5} = 1216^k$$

Determine column shears using dead load plus elastic earthquake results.

$$DL + 1.0\ EQ_L + 0.3\ EQ_T$$

$$V_{ux}^{DL+EQ} = \frac{(1150 + 2856 + 360 + 3141)}{20.0} = 375^k$$

$$V_{uy}^{DL+EQ} = \frac{(1845 + 4695 + 159 + 5150)}{20.0} = 592^k$$

$$V_u^{DL+EQ} = (375^2 + 592^2)^{0.5} = 701^k$$

$$DL + 0.3\ EQ_L + 1.0\ EQ_T$$

$$V_{ux}^{DL+EQ} = \frac{(1150 + 8355 + 360 + 9889)}{20.0} = 988^k\ *$$

$$V_{uy}^{DL+EQ} = \frac{(1845 + 1707 + 159 + 1849)}{20.0} = 278^k\ *$$

$$V_u^{DL+EQ} = (988^2 + 278^2)^{0.5} = 1026^k$$

$$P_{BOT\ COL}^{DL+EQ} = 914 + 993 = 1907^k$$

$M_{px\ BOT\ COL} = 9100 < (360 + 9889)$ ft-kips

$M_{py\ BOT\ COL} = 2000 \sim (159 + 1849)$ ft-kips

(Probable plastic moments associated with $P_{BOT\ COL}^{DL+EQ}$)

*The elastic moment value for $M_{x\ BOT\ COL}$ of (360+9889) would not quite be reached but say close enough for shear determination.

Comparison of shear force from elastic analysis with the shear forces associated with column yielding indicates that the shear force from the elastic earthquake analysis is least critical.

$\therefore V_{u\ DESIGN} = 1026^k$

The above comparison of shear forces also illustrates an undesireable aspect as far as seismic performance is concerned of the use of flared columns at short multi-column bents, which aspect is the potential for a high demand for shear resistance.

Transverse column reinforcement

Shear

$V_u = 1026^k$

Associated axial force

$$P_{\substack{DL+EQ \\ BOT\ COL}} = 914 \pm 993 = \begin{matrix} 1907^k \text{ max} \\ -79^k \text{ min} \end{matrix}$$

Because the column axial force can be a tensile force, the total shear should be resisted by shear reinforcement.

Because plastic hinging may occur at the bottom of the column, use the core section of the basic column section for shear resistance.

$$V_{s\ req'd} = \frac{V_u}{\phi} - V_c = \frac{1026}{0.85} - 0 = 1207^k$$

$$b_{w\ core} = 66.0 - 4.0 = 62.0"$$

$$d_{core} = 0.8 \times 62.0 = 49.60"$$

$$v_u = \frac{V_u}{\phi b_w d} = \frac{1026}{0.85 \times 62.0 \times 49.60} = 0.392 \text{ ksi}$$

$$v_u - v_c = 0.392 - 0 = 0.392 \text{ ksi}$$

$$(v_u - v_c)_{max\ allow} = 8\ (f'_c)^{0.5} = 8\ \frac{(3250)}{1000}^{0.5} = 0.456 \text{ ksi}$$

$$0.392 < 0.456 \text{ ok}$$

Try concentric #5 spirals at 3" pitch

$$V_s = \frac{A_v f_y d}{S} = \frac{4 \times 0.31 \times 60.0 \times 49.60}{3.0} = 1230^k > 1207^k \text{ ok}$$

Concentric spirals require revised arrangement of longitudinal column reinforcement.

Try 18-#9 full length in outer and inner ring plus 18-#9 for 2/3 the length of the top portion of the column in the outer ring.

Spacing at the bottom of the column

$$r_{outer\ ring} = 33.0 - 2.0 - 0.69 - 0.63 = 29.68"$$

$$\text{spacing} = \frac{2\pi \times 29.68}{18} = 10.36" > 8" \quad \text{N.G.}$$

See Article 1.5.11(A)

Try #8 longitudinal column reinforcement

24-#8 full length in outer and inner ring plus
24-#8 for 2/3 the length of the column at the top of the column in the outer ring.

$$A_{s\ TOP\ COL} = (48 + 24) = 72 - \#8 = 56.88 \text{ sq. in.} \sim 54.0 \text{ sq. in. ok}$$

$$A_{s\ BOT\ COL} = 48 - \#8 = 37.92 \text{ sq. in.} \sim 36.0 \text{ sq. in. ok}$$

Spacing at bottom of column

$$r_{outer\ ring} = 33.0 - 2.0 - 0.69 - 0.57 = 29.74"$$

$$\text{spacing} = \frac{2\pi \times 29.74}{24} = 7.79" < 8.0" \quad \text{ok}$$

Spacing at top of column of outer ring

$$\text{Spacing} = \frac{7.79}{2} = 3.9"$$ < than the preferred minimum spacing, therefore bundle partial length bars to the full length bars in the outer ring.

Confinement

Satisfy the following volumetric equations. Equations (2) and (3) apply to regions of potential plastic hinging only.

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f'_c}{f_y} \qquad \text{Eq. (1)}$$

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f'_c}{f_y} \left(0.5 + 1.25 \frac{Pe}{f'_c A_g}\right) \qquad \text{Eq. (2)}$$

$$\rho_s = 0.12 \frac{f'_c}{f_y} \left(0.5 + 1.25 \frac{Pe}{f'_c A_g}\right) \qquad \text{Eq. (3)}$$

The confinement requirements for the basic section at the bottom of the column will satisfy the requirements for the remainder of the column.

$$A_g = \frac{\pi (66.0)^2}{4} = 3421.2 \text{ sq. in.}$$

$$A_c = \frac{\pi (62.0)^2}{4} = 3019.1 \text{ sq. in.}$$

$f'_c = 3250$ psi , $f_y = 60000$ psi

$P_{e\ max} = 1907^k$ (dead load plus elastic earthquake results)

Eq. (1) $\rho_s = 0.45 \left(\frac{3421.2}{3019.1} - 1\right) \frac{3250}{60000} = 0.003246$

Eq. (2) $\rho_s = 0.003246 \left(0.5 + 1.25 \frac{1907 \times 1000}{3250 \times 3421.2}\right)$

$= 0.003246 \times 0.7144 = 0.00232$

Eq. (3) $\rho_s = 0.12 \frac{3250}{60000} \times 0.7144 = 0.00464$ controls <-------

Try 2 concentric #5 spirals at 3" pitch

$Vol._{spiral} = 0.31\pi \times 2 (33.0 - 2.0 - 0.35) = 59.70$

$+ 0.31\pi \times 2 (33.0 - 2.0 - 0.69 - 1.13 - 0.35) = \underline{56.15}$

115.85 cu.in.

$$Vol._{concrete} = 3019.1 \times 3.0 = 9057.3 \text{ cu.in.}$$

$$\rho_s = 115.85/9057.3 = 0.0128 > 0.00464 \quad \text{ok}$$

Use 2 concentric #5 spirals at 3" pitch for the full length of column and extend into the bent cap and footing.

Piles:

A preliminary determination of the footing size required using 70 ton piles indicated that this was not a practical solution considering the 18' column spacing. A common footing using 70 ton piles could present a practical solution, but in order to illustrate a footing design for individual footing, 100 ton piles will be used.

Ultimate bearing capacity = 400^k
Ultimate uplift capacity = 200^k
Ultimate lateral resistance = 40^k except for Group VII = 55^k

Footing:

$f'_c = 3250$ psi
$f_y = 60000$ psi

Determine the pile layout, footing size, and footing reinforcement required to resist the bottom of column forces and moments. Use the ℄ of bent (X-axis) and the ℄ of column (Y-axis) as the principal axes of the footing.

Minimum footing thickness

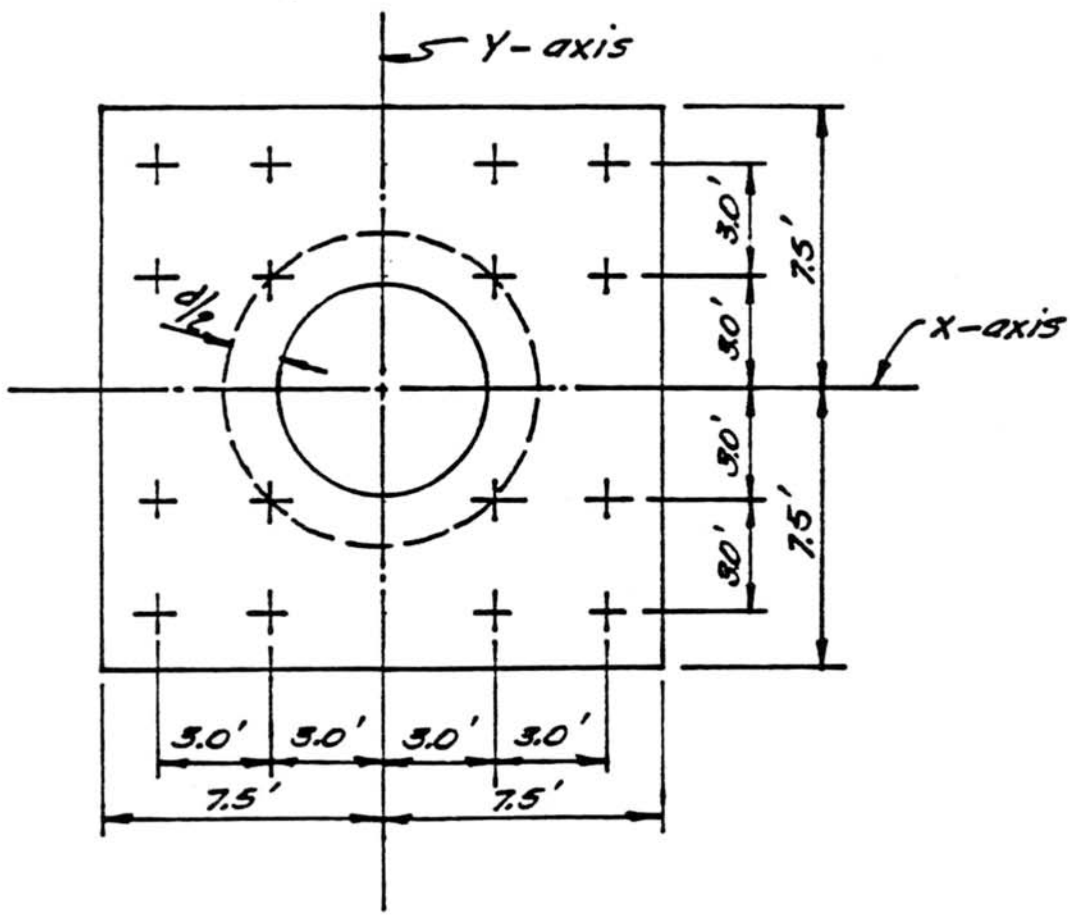
19.80"	development of outer ring of #8 column reinforcement
6.00"	additional embedment of inner ring of column reinforcement
3.26"	#11 bottom footing reinforcement
6.00"	clearance to bottom footing reinforcement
35.06"	

Determine a pile layout that is adequate for Group VII loads and check for the other group loads.

Group VII Bottom of column loads

Case	P	M_x	M_y	
1	914^k	7900 ft-kips	--- ft-kips	
2	3014	---	10400	Yielding of
3	-1186	---	3000	column
4	2104	6800	6800	
5	-276	3800	3800	
6	1396	5309	3501	DL + EQ
7	432	5309	3501	elastic
8	1907	2008	10249	analysis
9	-79	2008	10249	

Try 15' x 15' x 3.5' footing w/16 piles



A_{piles} = 16 piles

I_{piles} (each direction) $= 8\,(6.0)^2 = 288$
$= 8\,(3.0)^2 = \underline{72}$
360 pile-ft.^{2}

$P^{DL}_{ftg} = 15.0 \times 15.0 \times 3.5 \times 0.150 = 118$

$P^{DL}_{cover} = (225.0 - 23.8)\ 2 \times 0.120 = \underline{48}$
166^k

Pile reactions - Group VII loading

Case 1

$$P = \frac{(914 + 166)}{16} \pm \frac{7900 \times 6}{360}$$

$$= 67.5 \pm 131.7 = 199.2^k \text{ max.} \quad -64.2^k \text{ min.}$$

Case 2

$$P = \frac{(3014 + 166)}{16} \pm \frac{10400 \times 6}{360}$$

$$= 198.8 \pm 173.3 = 372.1^k \text{ max.} \quad 25.5^k \text{ min.}$$

Case 3

$$P = \frac{(-1186 + 166)}{16} \pm \frac{3000 \times 6}{360}$$

$$= -63.8 \pm 50.0 = -13.8^k \text{ max.} \quad -113.8^k \text{ min.}$$

Case 4

$$P = \frac{(2104 + 166)}{16} \pm \frac{6800 \times 6}{360} \pm \frac{6800 \times 6}{360}$$

$$= 141.9 \pm 113.3 \pm 113.3 = 368.5^k \text{ max.} \quad -84.7^k \text{ min.}$$

Case 5

$$P = \frac{(-276 + 166)}{16} \pm \frac{3800 \times 6}{360} \pm \frac{3800 \times 6}{360}$$

$$= -6.9 \pm 63.3 \pm 63.3 = 119.7^k \text{ max.} \quad -133.5^k \text{ min.}$$

Case 6

$$P = \frac{(1396 + 166)}{16} \pm \frac{5309 \times 6}{360} \pm \frac{3501 \times 6}{360}$$

$$= 97.6 \pm 88.5 \pm 58.4 = 244.5^k \text{ max.} \quad -49.3^k \text{ min.}$$

Case 7

$$P = \frac{(432 + 166)}{16} \pm 88.5 \pm 58.4$$

$$= 37.4 \pm 88.5 \pm 58.4 = 184.3^k \text{ max.} \quad -109.5^k \text{ min.}$$

Case 8

$$P = \frac{1907 + 166)}{16} \pm \frac{2008 \times 6}{360} \pm \frac{10249 \times 6}{360}$$

$$= 129.6 \pm 33.5 \pm 170.8 = 333.9^k \text{max.} \quad -74.7^k \text{min.}$$

Case 9

$$P = \frac{(-79 + 166)}{16} \pm 33.5 \pm 170.8$$

$$= 5.4 \pm 33.5 \pm 170.8 = 209.7^k \text{ max.} \quad -198.9^k \text{ min.}$$

The pile layout satisfies load cases for DL + EQ from an elastic analysis and also satisfies load cases from the yielding of the columns. For internal footing design use only the load cases from an elastic analysis.

Check pile layout for other group loads.

Factored Group I loads (at bottom of column)

Case 1

$$P = 1.3\ (914 + \frac{1.67}{1.21} \times 256) = 1648^k$$

$$M_x = 1.3\ (159 + \frac{1.67}{1.21}\ 159) = 492 \text{ ft-kips}$$

$$M_y = 1.3\ (360 + \frac{1.67}{1.21}\ 3) = 473 \text{ ft-kips}$$

Case 2

$$P = 1.3\ (914 + \frac{1.67}{1.21}\ 131) = 1423^k$$

$$M_x = 1.3\ (159 + \frac{1.67}{1.21}\ 756) = 1563 \text{ ft-kips}$$

$$M_y = 1.3\ (360 + \frac{1.67}{1.21}\ 2) = 472 \text{ ft-kips}$$

Case 3

$$P = 1.3\ (914 + \frac{1.67}{1.21}\ 130) = 1421^k$$

$$M_x = 1.3\ (159 + \frac{1.67}{1.21}\ 95) = 377 \text{ ft-kips}$$

$$M_y = 1.3\ (360 + \frac{1.67}{1.21}\ 188) = 805 \text{ ft-kips}$$

Case 4

$$P = 1.3\ (914 + \frac{576}{1.21}) = 1807^k$$

$$M_x = 1.3\ (159 + \frac{399}{1.21}) = 635 \text{ ft-kips}$$

$$M_y = 1.3\ (360 + \frac{102}{1.21}) = 578 \text{ ft-kips}$$

Case 5

$$P = 1.3\ (914 + \frac{383}{1.21}) = 1600^k$$

$$M_x = 1.3\ (159 + \frac{1746}{1.21}) = 2083 \text{ ft-kips}$$

$$M_y = 1.3\ (360 + \frac{80}{1.21}) = 554 \text{ ft-kips}$$

Case 6

$$P = 1.3\ (914 + \frac{450}{1.21}) = 1672^k$$

$$M_x = 1.3\ (159 + \frac{335}{1.21}) = 567 \text{ ft-kips}$$

$$M_y = 1.3\ (360 + \frac{287}{1.21}) = 776 \text{ ft-kips}$$

Pile reactions - Factored Group I loading
By inspection Case 4 and Case 5 will produce the maximum pile reaction.

Case 4

$$P = \frac{(1.3 \times 166 + 1807)}{16} \pm \frac{635 \times 6}{360} \pm \frac{578 \times 6}{360}$$

$$= 126.4 \pm 10.6 \pm 9.6 = 146.6^k \text{ max}$$
$$< 0.75 \times 400 = 300^k \quad \text{ok}$$

Case 5

$$P = \frac{(1.3 \times 166 + 1600)}{16} \pm \frac{2083 \times 6}{360} \pm \frac{554 \times 6}{360}$$

$$= 113.5 \pm 34.7 \pm 9.2 = 157.4^k \text{ max} < 300^k \quad \text{ok}$$

Determine footing shear requirements

Equivalent square column section

$$h^2 = \frac{\pi (66.0)^2}{4} = 3421.2 \text{ ; } h = 58.5" \ (4.87')$$

From a comparison of pile reactions it can be determined that either Group VII Case 8 or Case 9 loading will control, because of the Group VII loading cases, those from the elastic analysis of DL+EQ are the lesser. See Article 1.2.20(F).

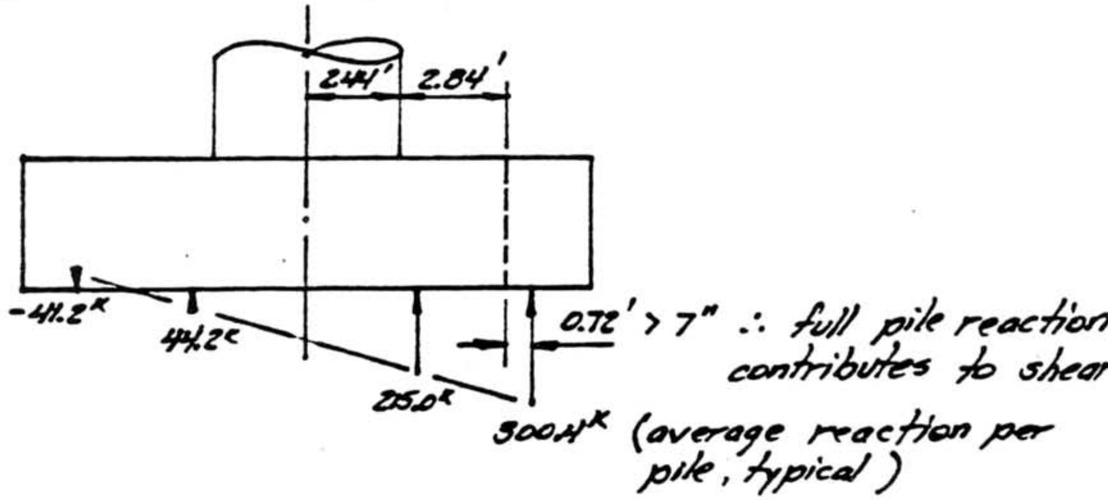
Assume piles are HP14 x 89 for determination of contribution of a pile reaction to the shear on a particular section through the footing.

Assume #9 footing reinforcement

$$d_{min}^{\#9} = 42.0 - 6.0 - 1.5 \times 1.25 = 34.12" \ (2.84')$$

Shear at section through footing at distance d from face of column and parallel to the Y-axis.

Group VII Case 8 loading



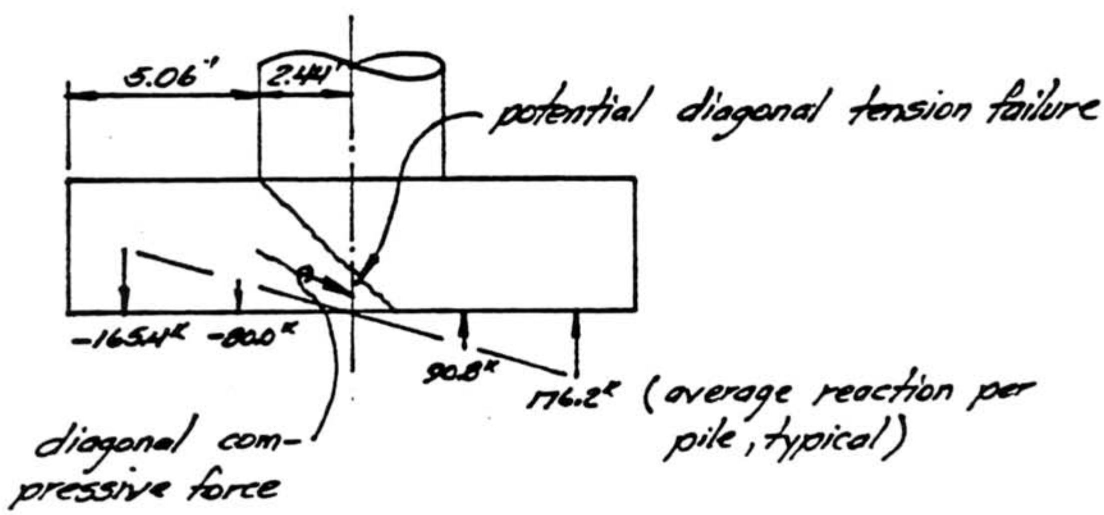
$$V_u \sim 4 \times 300.4 = 1202^k$$

$$\phi V_c = 0.85 \times 2 \frac{(3250)^{0.5}}{1000} (15.0 \times 12)\ 34.12 = 595^k < 1202^k$$

∴ need shear reinforcement or thicker footing

Shear at section through footing at face of column and parallel to the Y-axis.

Group VII Case 9 loading



$$
\begin{aligned}
V_u \sim\ & 4 \times 165.4 && = 662 \\
& 4 \times 80.0 && = 320 \\
& 15.0 \times 5.06 \times 3.5 \times 0.150 && = 40 \\
& 15.0 \times 5.06 \times 2.0 \times 0.120 && = \underline{18} \\
& && 1040^{k}
\end{aligned}
$$

Need to pick up vertical component of diagonal compressive force at the bottom of the footing with vertical reinforcement in order to transfer this force to the top of footing so it can be transferred into the column area and be picked up by the longitudinal column reinforcement. Assumed this load condition could occur about each axis of the footing.

$$A_s \text{ req'd} \sim \frac{1040}{0.9 \times 60} = 19.26 \text{ sq. in.}$$

using #5 [

6 rows @ 12" x 6" spacing, $A_s = 2 \times 6 \times 6 \times 0.31 = 22.32$ sq. in. ok
(Y-axis)

3 rows @ 12" x 6" spacing, $A_s = 2 \times 3 \times 11 \times 0.31 = 20.46$ sq. in. ok
(X-axis)

Shear at section through footing at distance d/2 from perimeter of column.

Group VII Case 8 loading

$d/2 = 2.84/2 = 1.42'$

disregard tensile pile reactions and dead load of footing and cover.

$$
\begin{aligned}
V_u \sim\ & 4 \times 300.4 && = 1202 \\
& 2 \times 215.0 && = 430 \\
& 2 \times 0.5 \times 215.0 && = 215 \\
& 2 \times 44.2 && = 88 \\
& 2 \times 0.5 \times 44.2 && = \underline{44} \\
& && 1979^{k}
\end{aligned}
$$

$$b_o = 2\pi\ (2.75 + 1.42) = 26.2' = 314.4"$$

$$\phi V_c = 0.85 \times 4 \frac{(3250)^{0.5}}{1000}\ 314.4 \times 34.12 = 2079^{k} > 1979^{k} \quad \text{ok}$$

Using 3.5' thick footing determine shear reinforcement required.

Group VII Case 8 loading

$$V_{s\ req'd} = \frac{1202}{0.85} - \frac{595}{0.85} = 714^k$$

try #5 @ 12 each way

$$V_s = \frac{15 \times 0.31 \times 60.0 \times 34.12}{12} = 793^k > 714^k \quad \text{ok}$$

Group VII Case 9 loading

#5 [@ 12 each way ok by comparison

Determine stirrup layout after flexural reinforcement is determined.

Determine flexural reinforcement required.

Bottom of footing flexural reinforcement.
Section at face of column.

Group VII Case 8 loading will control

$$M_u \sim \begin{array}{rl} 4 \times 300.4\ (6.0 - 2.44) = & 4278 \\ 4 \times 215.0\ (3.0 - 2.44) = & \underline{482} \\ & 4760 \text{ ft-kips} \end{array}$$

$$1.2\ M_{cr} = 1.2 \times 7.5\ (f'_c)^{0.5}\ \frac{I_g}{y_t}$$

$$= 1.2 \times 7.5\ \frac{(3250)^{0.5}}{1000} \times \frac{1/12(15.0 \times 12)(42.0)^3}{0.5 \times 42.0\ (12)} = 2263 \text{ ft-kips}$$

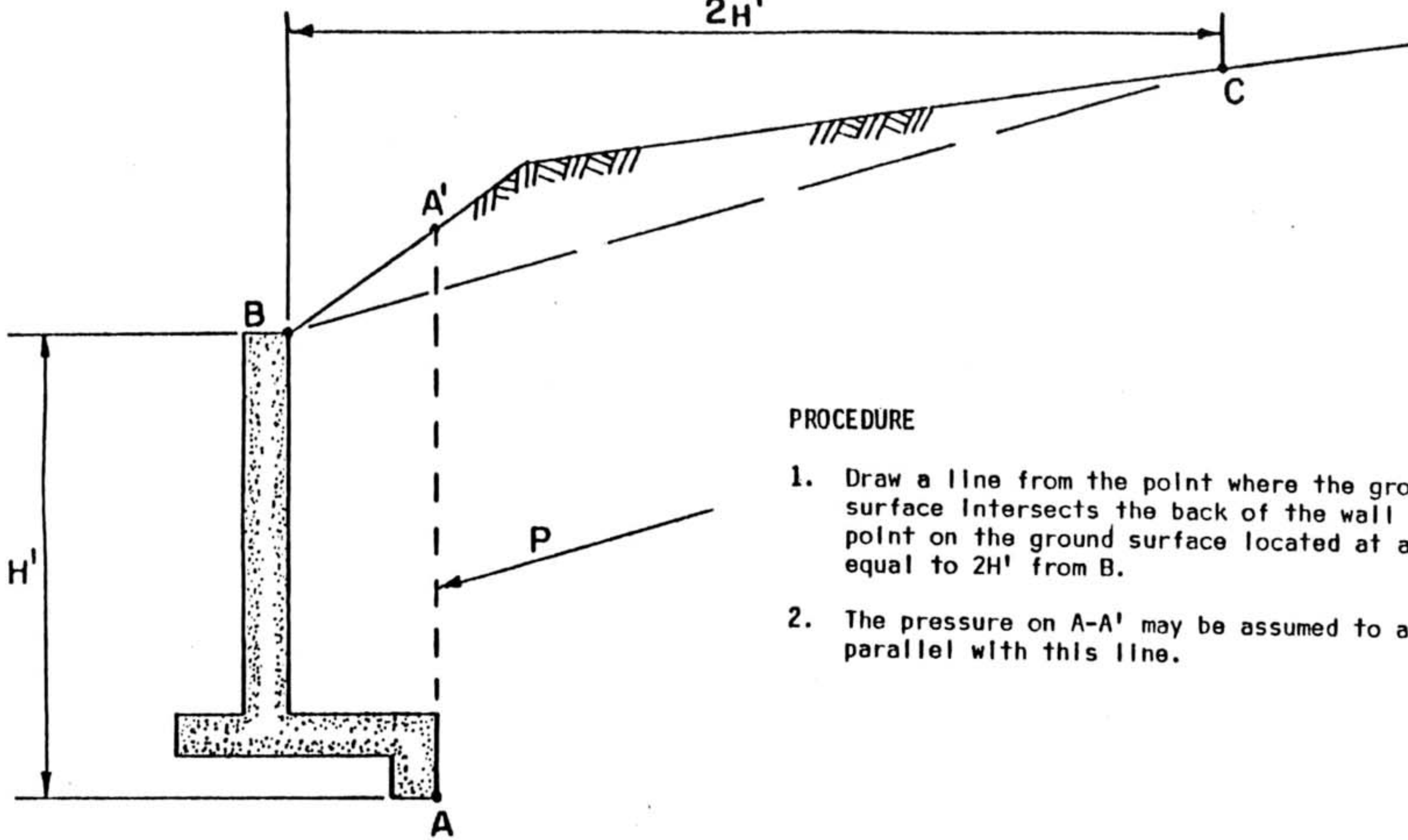
$$M_u > 1.2\ M_{cr} \quad \therefore A_{s\ req'd.} > A_{s\ min.}$$

try A_g = #9 @ 6 $\pm$ = 31 x 1.0 = 31.0 sq. in.

$$a = \frac{31.0 \times 60.0}{0.85 \times 3.25\ (15.0 \times 12)} = 3.74"$$

$$\phi M_n = 1.0 \times 31.0 \times 60.0\ (34.12 - \frac{3.74}{2})\ /\ 12 = 4999 \text{ ft-kips}$$
$$> 4760 \text{ ft-kips} \quad \text{ok}$$

$$\rho_s = 31.0/(15.0 \times 12 \times 34.12) = 0.0050$$



PROCEDURE

1. Draw a line from the point where the ground surface intersects the back of the wall (B) to a point on the ground surface located at a distance equal to 2H' from B.

2. The pressure on A-A' may be assumed to act parallel with this line.

APPROXIMATE METHOD FOR DIRECTION OF RANKINE EARTH PRESSURE

Figure 3-4

$$P_{pile} = \frac{1187}{16} \pm \frac{238 \times 6}{360} \pm \frac{515 \times 6}{360}$$

$$= 74.2 \pm 4.0 \pm 8.6 = 86.8^k \text{ max}$$

Case 2 controls, check section at face of column.

$$\begin{aligned} M \sim\ & 4 \times 87.3\ (6.0 - 2.44) = 1243 \\ & 4 \times 80.8\ (3.0 - 2.44) = \underline{181} \\ & \qquad\qquad\qquad\qquad 1424 \text{ ft-kips} \end{aligned}$$

b = 180.0"
d = 34.12"
n = 9

f_s = 17.69 ksi < 24.0 ksi ok

f_c = 0.688 ksi

Use #9 @ 6 $\pm$ total 31 each direction

Top of footing flexural reinforcement
Check section at face of column

Group VII Case 9 loading

$$\begin{aligned} M_u \sim\ & 4 \times 165.4\ (6.0 - 2.44) &&= 2355 \\ & 4 \times 80.0\ (3.0 - 2.44) &&= 179 \\ 5.06 \times 15.0 & \times 3.5 \times 0.150 \times 2.53 &&= 101 \\ 5.06 \times 15.0 & \times 2.0 \times 0.120 \times 2.53 &&= \underline{46} \\ & &&\ \ 2681 \text{ ft-kips} \end{aligned}$$

$$d_{min}^{\#9} = 42.0 - 3.0 - 1.5 \times 1.25 = 37.12" \ (3.09')$$

$$1.2\ M_{cr} = 1.2 \times 7.5\ \frac{(3250)^{0.5}}{1000}\ \frac{1/12(15.0 \times 12)(42.0)^3}{0.5 \times 42.0(12)} = 2263 \text{ ft-kips}$$

$M_u > 1.2\ M_{cr}\ \therefore\ A_{s\ req'd} > A_{s\ min}$

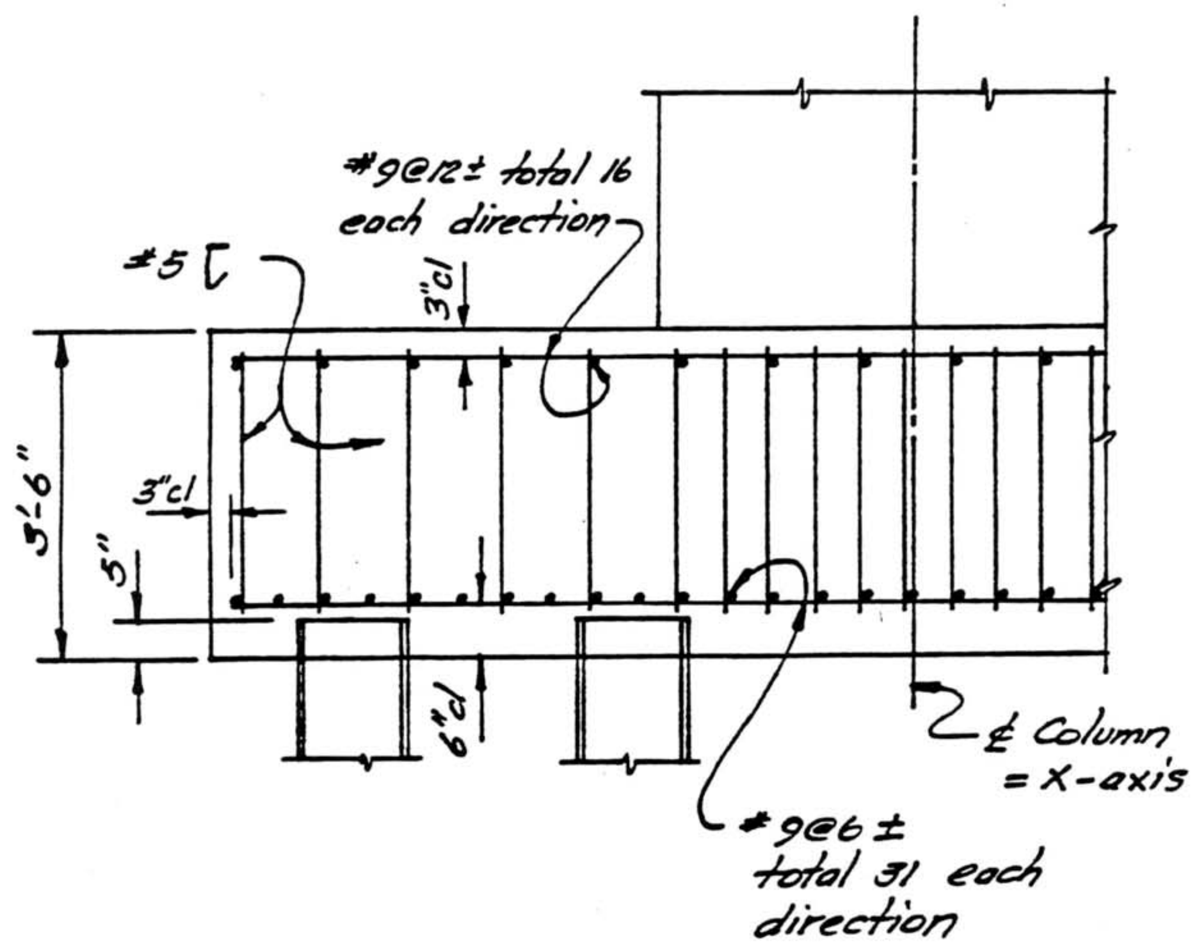
try A_s = #9 @ 12 $\pm$ = 16 x 1.0 = 16.0 sq.in.

$$a = \frac{16.0 \times 60.0}{0.85 \times 3.25\ (15.0 \times 12)} = 1.93"$$

$$\phi M_n = 1.0 \times 16.0 \times 60.0\ (37.12 - \frac{1.93}{2})\ /\ 12 = 2892 \text{ ft-kips}$$

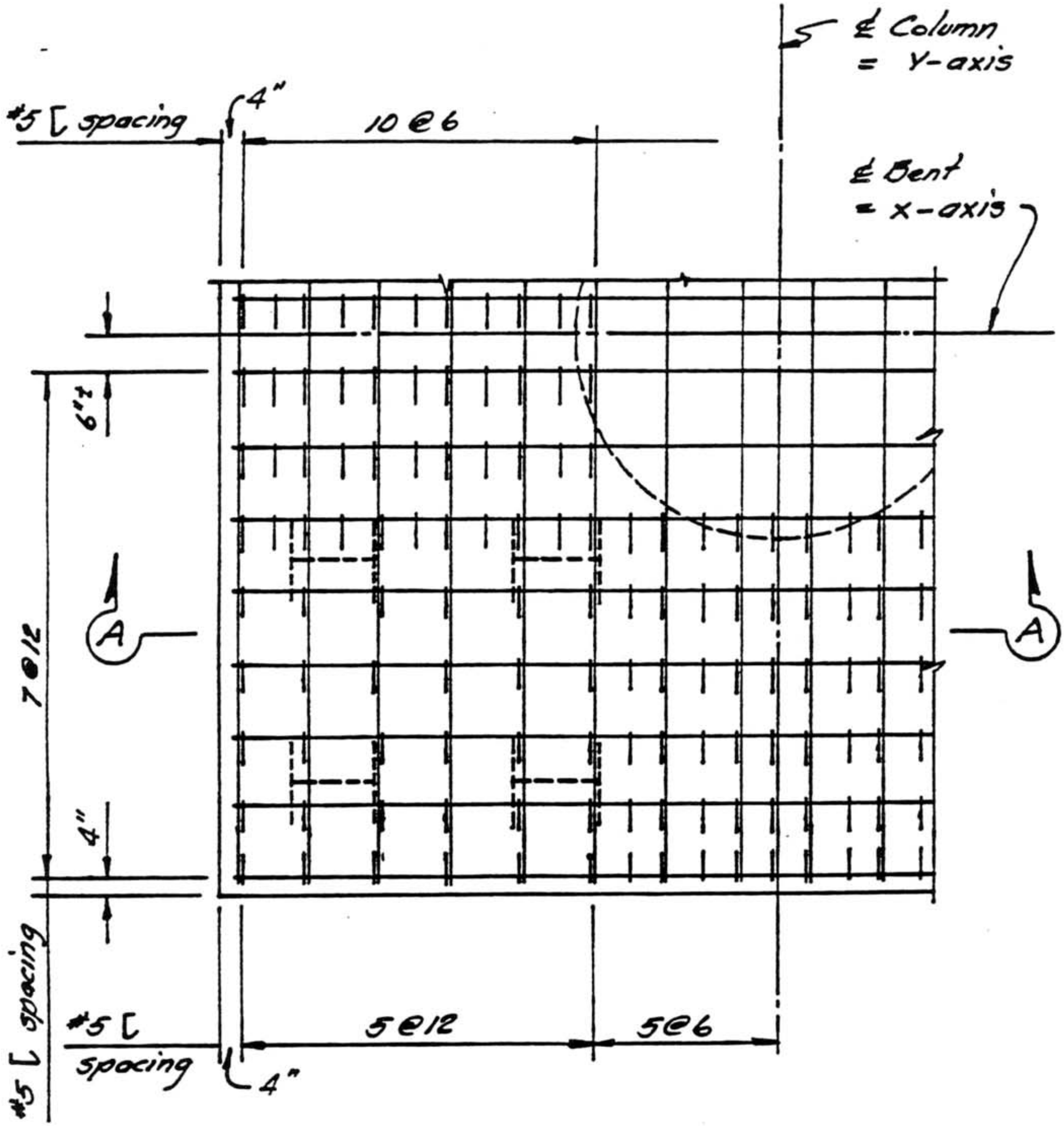
2681 ft-kips < 2892 ft-kips ok

Use #9 @ 12 ± total 16 each direction



SECTION A-A

½" = 1'-0"



Note: Details are symmetrical about ℄ Column and ℄ Bent.

PART FOOTING PLAN

1/2" = 1'-0"